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MINUTES OF PROCEEDINGS
OF
THE INSTITUTION
OF
CIVIL ENGINEERS;
WITH OTHER
SELECTED AND ABSTRACTED PAPERS.

VOL. CLXXXIV.

EDITED BY
J. H. T. TUDSBERY, D.Sc., M. INST. C.E., SECRETARY.

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CORRIGENDA.

- Vol. clxxxiii, p. 252, l. 8, *for* "rock" *read* "yard."
- " " " 253, l. 5 from bottom, *insert* "centrifugal" *before* "type."
- " " " 258, l. 7, *insert* "and was" *before* "probably caused."
- " " " 261, l. 1, *for* "in blocks" *read* "in the blocks."
- " " " 261, l. 2 from bottom, *for* "foundation" *read* "framing."
- " " Plate 8, Fig. 1, *for* "Western Tibers" *read* "Western Tiers,"
and for "Arthur's Lake" *read* "Arthur's Lakes."
- " clxxxiv, p. 211, l. 22, *for* "this" *read* "the entrance."
- " " " 211, l. 23, *delete* "an entrance."

THE
INSTITUTION
OF
CIVIL ENGINEERS.

SESSION 1910-1911.—PART II.

SECT. I.—MINUTES OF PROCEEDINGS.

20 December, 1910.

ALEXANDER SIEMENS, President,
in the Chair.

(Paper No. 3885.)

“The Winning of Coastal Lands in Holland.”

By ALFRED EDWARD CAREY, M. Inst. C.E.

THE foreshore-defence works on the Dutch coast have received but scanty notice in the Proceedings of The Institution, and indeed little technical literature dealing with such work exists in the English language. In this Paper the Author presents a summary of the observations made by him during a recent inspection of the Dutch coast-line from Flushing to as far north as Texel, when he had the advantage of access to official sources of information. Some comparison of British and Dutch practice is also attempted. The Dutch engineers were the pioneers of land-reclamation in England, their work in the fens commencing about 1652.

The motto of the province of Zeeland, *Luctor et emergo*, is an apt summary of the struggle of the Dutch people. The country being primarily agricultural, the great works which have remodelled the map of Holland are necessarily based upon economical engineering methods and devices, which are the result of slow evolution.

HISTORICAL.

No description of the Dutch coastal regime would be complete without some reference to its historical growth. Napoleon described Holland as the “alluvion of French rivers.” When the Roman forces penetrated to the North Sea, Tacitus and Pliny, in noting the precarious tenure of the half-submerged lands which now constitute the kingdom of Holland, recorded their wonder at the

[THE INST. C.E. VOL. CLXXXIV.]

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diurnal tides, a phenomenon novel to dwellers on the shores of the Mediterranean. Roman travellers were, they said, in doubt if the country were land or water. Eumeneus spoke of lands which trembled under the tread of the pedestrian, and seemed to consist of a shell of land floating over water. The latest geological view of the conformation of the overlying strata of Holland presents a singular analogy to this remark. The late Mr. Harrison Hayter, Past-President Inst. C.E., stated¹ that on a site originally intended for one of the protective dams for the Amsterdam ship-canal works, two men with a hand-auger bored to a depth of 50 feet in 20 minutes.

Travellers in Holland cannot fail to notice the isolated mounds, about 30 to 40 feet high, which occur in many parts of the country, but are most frequent in the province of Groningen. The Dutch call them *terpen*, or *wierden*. The early habitations of settlers were on these artificial mounds; and the Emperor Vitellius (A.D. 69) is reputed to have taken refuge on one. The soil of which they are composed is now often used as a top dressing for the adjoining lands, and when the mounds are levelled, they frequently prove to be museums of antiquities going back to the Stone Age.

One-third only of the area of Holland to-day (i.e., the south-east part of the country) is more than 1 metre above the level of average high water; the remainder of the country is practically below that level. As early as the twelfth century, the Dutch were renowned for their skill in coastal defence; thus, in 1179, after great inundations, Dutch workmen were called in to construct protective works for Bruges, the groynes then built consisting of fascines and earthworks. A century later, the practice was to hold down the fascines with piling or heavy beams of timber; but the destruction of such timber by the *paalworm* or teredo—said to have reached northern Europe in ships engaged in the eastern trade—resulted in the gradual introduction of masonry. The earliest efforts of the scattered settlers were directed to the maintenance of existing lands, but reclamation work was soon commenced. The embankment of the rivers, and the winning of clay lands formed by the deposition of ooze, has gone on thus for centuries. Areas of alluvion so created are termed *polders* in Flanders and in the provinces of North and South Holland and Zeeland, and *kwelders* in the provinces of Friesland and Groningen. In Essex such areas are called “levels”; and it may be noted incidentally as a curious fact that in Essex the word “dike” means a ditch or artificial open drain, and in Holland

¹ Minutes of Proceedings Inst. C.E., vol. lxii, p. 8.

the same word *dijk* indicates a dam or embankment, thus corresponding with the French *digue*.

The sequence of the national embanking operations may be summarized thus:—

In 1277 the country near the mouth of the Ems was inundated, and for more than 200 years the flooded land remained a swamp. The struggle to reclaim these lands, which has been going on from the middle of the sixteenth century to the present day, has resulted in the recovery from the sea of more than 12,500 acres on the coast of Friesland, and of about 57,000 acres on the coast of the province of Groningen. The winning of lands in this district is still proceeding steadily. In the centre of Friesland, an inland sea having an area of 25,000 acres formerly existed, and in Roman times the river IJssel flowed through this lake to the sea. Between the thirteenth and sixteenth centuries enormous embankments were constructed; but in the seventeenth century these embankments were found to be too low, as they were often breached, in spite of the fact that the coast-line is protected on the north-west by a chain of islands.

In November, 1750, a violent tempest from the north-west destroyed the dams in several places and flooded the country as far as the city of Groningen, causing the death of 20,000 persons. Much of this flooded land has since been reclaimed, but probably the area of the Zuider Zee was considerably enlarged by that historic inundation.

In prehistoric times the Rhine must have followed many channels in debouching on the North Sea, and the gaps which occur in the line of sand-dunes along the coast to-day are doubtless due to the varying outlets of the river. Where sand-dunes exist, they form a natural barrier of defence, and one important branch of the art of the Waterstaat engineers in Holland, who deal with these matters, is the maintenance of the dunes. The measures adopted with this object will be described later.

Before the seventeenth century the country round Alkmaar and Haarlem was inundated when excessively high tides occurred, but between 1540 and 1648 about 63,000 acres in this district were reclaimed. The lake of Haarlem was originally a vast swamp intersected by small rivers, its area of about 45,000 acres being continually increased by the flooding of fresh lands. In 1840, however, further drainage-operations were decided on, and a sum of about £670,000 was voted by the State. The estimate for the drainage of this area was about £17 per acre; the actual cost of the work amounted to only about £14 per acre.

In 1421 the dam near Werkendam, between Dordrecht and Geertruidenberg, was broken down, and 20,000 people perished. The whole of this area to-day is under cultivation, with the exception of the Biesbosch district, which consists of a marshy archipelago, forming one of the best fowling-districts in Holland. The island of Overflakkee consisted, in the fifteenth century, of three islands which were joined by embankments, and in 1751 they were united with the island of Goeree. These four islands originally had an area of 10,000 acres; to-day it is 60,000 acres. The island of Walcheren, again, consisted originally of dozens of small marshy islands. These were united, by the construction of dams, at the end of the fourteenth century, and on that island to-day is the famous dam of Westkapelle, close alongside of which runs a deep channel for ocean-going steamers. The reclamations effected at the mouths of the Maas and Scheldt since the twelfth century have increased the area of Holland by about 250,000 acres of agricultural land.

POLDERS.

Before describing in detail the foreshore-defences of the Dutch coast, it may be desirable to state the general conditions regulating this class of work.

In Holland the floods pasture on alluvial lands and follow the tide as it recedes. By slow degrees soil accretes, and the growth of vegetation fixes the ooze, film on film, as it washes across the flats. When such land is judged to have reached maturity, it is enclosed by dams, and another polder comes into existence. With the object of draining such a tract of land, it is first entirely isolated by embankments, and a ditch or canal is dug around it, inside these banks. An accumulating-reservoir is necessary to enable discharge to take place at low tide; and, depending on the range of tide, the level at which water stands in these reservoirs is a measure of their maximum efficiency in discharge. In passing from the south to the north of Holland the range of tide decreases, and the size of the accumulating-reservoirs also diminishes, which appears paradoxical. It is explained, however, by the use made of the navigation-canals as channels of discharge, and by the existence of natural reservoirs under and at the base of the sand-dunes. This point is examined in some detail later in the Paper (p. 6). Where navigation-canals exist, the interior water is pumped into them, either wind or steam power being employed for driving the pumps. When a tract of land is thus roughly cleared of water, swamps result, and ditches

are laid out intersecting the area like the lines on a chess-board. The purpose of these ditches is twofold: they provide superficial drainage, and the fertile soil removed in cutting them is spread over the surface of the land to be reclaimed.

Ultimately sluices are constructed: by means of these and wind-mill pumps the level of the flood-water is regulated, and the polder becomes fit for agricultural purposes. Such an operation as that just described obviously requires considerable time, but is usually practicable at small expense. When once the windmill pumps and the sluices are installed, the cost of maintaining the desired water-level is trifling.

POLDER-BOARDS.

As is the case in the low-lying districts of England, the regulation and control of the sea-defences are in the hands of bodies of commissioners. In England such commissioners are nominated by the Crown. They have to be owners of real estate of a given rateable value in the county concerned, but are not necessarily locally interested in the particular area affected. These bodies of commissioners, whose authority in several cases dates from the Tudor period, have statutory rights for the levying of a scot or charge per acre on a defined area of land subject to flooding. The commissioners hold periodical courts, at which the engineer in charge of the levels makes his recommendations for the ensuing season. The court issues orders to landowners to carry out the work considered necessary, and any landowner failing to comply is subject to penalties. One or more resident marsh-bailiffs are in charge of a given level. On some levels the commissioners themselves carry out the orders of the court, charging each landowner affected his proportion of the cost of the work.

The Dutch system differs materially from this. The area known as a *polder* is under the control of a Polder-Board, ten or more representative members being elected by the landowners. These, elected members, who form the general committee, elect a committee of five, which is termed the *daijli* committee of the polder, and the engineer for the district is responsible to the latter. These committees often have control over large funds, as some of the areas affected are extensive.

One dominant factor in the Dutch system as compared with the English is the fact that new polders are constantly being embanked and won. Numerous instances could be cited of Acts of Parliament passed in the reign of Queen Elizabeth, whereby various local bodies

in England were empowered to carry out the winning of waste lands overflowed by tidal waters. The depreciation in the value of land in England, more especially of the clay lands of Essex, has checked any attempt at enclosing fresh swampy tracts. In localities such as New England, where the sea-defences are broken down, no attempt is made to recover land so lost.

In Holland there is a constant attempt to extend the agricultural area of the State. In some places, such as parts of Walcheren, the cost of maintaining the sea-walls is greater than the locality can bear. In cases of this kind, which are called *calamiteuses polders*, the State and the Province bear either part of or all of the cost of the sea-defences. Where agriculture flourishes, the landowners defray the cost under scot or levy.

As showing how considerable must be the total expenditure in the kingdom of Holland under this head, it may be mentioned that the polder of Schouwen, the area of which is about 50,000 acres, has a budget, in respect of coastal defence, which is a minimum of about £17,000 per annum, and occasionally reaches £25,000. Protecting the administrative area of 22,500 acres there are about 18 miles of sea-walls and about $2\frac{1}{2}$ miles of sand-dunes. On the Dutch coast the total length of the sea-dikes is about 1,600 miles.

WATER-LEVELS.

A few words as to standard levels in Holland are necessary. The datum is known as *Amsterdamsch Peil*, marked "A.P." on Dutch maps. This represents mean-tide level at Ymuiden, and as a datum corresponds with the Ordnance datum of English maps, but is about $11\frac{1}{2}$ inches (0.30 metre) above it. The variation in tidal range on the Dutch coast is shown in Fig. 1, Plate 1, and will be seen to be considerable. All canal and inland waters are by law maintained artificially at -0.5 metre (-1.64 foot) A.P.

The object of the maintenance of an artificial water-level throughout the country by pumping is to provide what is in fact an immense drainage-reservoir for flood-water in case of inundation. Windmills are largely being superseded by steam-pumps, wind-power being capricious. It is stated that wind-power permits of continuous pumping during only 130 days in the year.

In view of the point already mentioned (p. 4) as to the size of the compensating reservoirs by which the outflow drainage from the polders is equalized, a very interesting collateral investigation has been made by Mr. J. M. K. Pennink.

The water-supply of the city of Amsterdam has, for a long period, been pumped from the sand-dunes. The questions whether this supply is likely to be permanent, and what are the physical causes which produce it, have been much debated. Mr. Pennink held the opinion that the reservoir of water existing in the base of the dunes was due to rain and condensation alone. This opinion, however, was not shared by German engineers, who believed that the supply was augmented by subterranean land-drainage,

The water-level in the dunes rises under capillary attraction and from external sea-pressure. The Dutch and Belgian engineers were at one in attributing the existence of the water-supply primarily to rainfall. Of the rainfall 64 per cent. is absorbed by the dunes, and the remaining 36 per cent. is lost by evaporation, absorption due to vegetation, etc. Mr. Pennink's theory is that a part of the 36 per cent. filters downwards, and escapes laterally. It would appear that only a small portion of the water finds its way directly into the sea.

The rainfall is greater on the dune areas than on any other portion of the country. Borings made by Mr. Hertzberg in the island of Norderney showed brackish water at a depth of 200 feet; above this level the water was fresh, and it was concluded that a substratum of salt water probably exists under Holland, thus bearing out, in an extraordinary manner, the remark of the Roman writer quoted in the early part of this Paper.

In 1902-3, when an additional source of water-supply for Amsterdam was urgently needed, the inadvisability of going too deep for the water was a paramount consideration, as the risk of drawing the underlying salt water into the upper natural reservoir, and rendering the whole supply brackish, was urged, the balance of pressures between the fresh, brackish and salt zones being very unstable. With a view to increase the supply, a proposal was made to divert part of the outflow from the irrigation-canals, causing some discharge from these into the dunes, but the quality of the canal-water was not good, and this would have converted the dunes into filter-beds which, in time, would have become inoperative.

By further comparison of the levels of the water in the substrata of the dunes, and of the fluctuations of these levels under pumping, it has been demonstrated that there exists a transition zone between the fresh-water and salt-water levels, and in this zone the water is brackish. Its depth below the intake in the dune is 33 to 66 feet.

A lowering of the fresh-water level under Amsterdam would, it is asserted, cause a corresponding and much greater drop in the

water-level of the dunes, owing to the balance of pressure now existing between the two. Such lowering, it is believed, would possibly break the fresh-water seal which the dunes naturally cause.

SAND-DUNES.

The geological conformation of the Netherlands affords a key to its hydrographical economy, the western and northern areas being alluvial, and the eastern and southern diluvial, the surface of the country here having been previously subjected to glacial denudation. Near the German frontier, geological formations older than the tertiaries crop out.¹ Here and there in the more recent deposits of Zeeland, North Friesland, and Groningen, the alluvium or sea-clay rises to a level of + 2 metres (6·6 feet) A.P. This alluvial deposit, however, is generally 1 to 2 metres, and in reclaimed lakes even 5 metres (16·4 feet), below sea-level. The chief feature of the alluvial base on which the superficial deposits of the country rest is the existence of peat, which was formed by the growth of bog plants on the swampy lands in rear of the dunes. This peat is thickest in the district of Utrecht and the province of North Holland, the thickness averaging about 15 feet, and in some places attaining 30 feet. Occasionally the peat exists at a level as low as - 19 metres (- 62 feet) A.P. Through this peaty swamp the rivers Rhine, Maas and Scheldt formerly forced their way to the sea by constantly shifting channels. At the same time they deposited the river-clays, which to-day form the most fertile soil of Holland. The remains of

¹ For a description of the sub-glacial strata of Holland and their correlation with the Crag beds of Suffolk, see F. W. Harmer, "On the Pliocene Deposits of Holland and their relation to the English and Belgian Crags, with a suggestion for the establishment of a new zone, 'Amstelien,' and some remarks on the geographical conditions of the Pliocene Epoch in Northern Europe" (Quarterly Journal of the Geological Society, vol. 52, 1896, p. 748). The depression in which the Pliocene strata of Holland rest is a shallow basin, the sides of which rise to the west, east and south towards Norfolk, Germany and Belgium respectively. A boring 1,098 feet in depth at Amsterdam did not reach these beds. A rise of 500-600 feet in the Pliocene sea-bed in the south of England and north-west of France was probably contemporaneous with a subsidence over the Dutch area of possibly more than 1,500 feet. The embouchure of the Rhine and its tributary rivers, after crossing the present site of the North Sea, appears to have struck the Essex coast at Walton, and thence flowed north into a sea-basin east and west of Cromer. From the geological point of view Holland is thus the alluvion of a system of vast rivers and estuaries, and its superficial deposits are the product of ice-borne debris from wide reas.—A. E. C.

Roman buildings, such as those at Katwijk, and near Domburg on the island of Walcheren, show that the protecting range of sand-dunes was formerly much wider, and probably of less elevation, than it is at present. It was also more continuous. The weakening of the line of defence afforded by the dunes, due partly to the action of the rivers, and partly to sea-storms, caused a penetration of the sea into the interior of Holland, and between the tenth and thirteenth centuries the Zuider Zee came thus into being. The principal spots in Holland where the protection of the dunes has broken down, and artificial protection has become necessary, are Westkapelle, on the island of Walcheren, where the sea-embankment, built in the fifteenth century, is about 3,800 yards long, and Hondsbosse and Petten, in North Holland, where the embankments, also constructed in the fifteenth century, extend for about 5,250 yards.

Up to the sixteenth century the dunes were continually changing, both in contour and in position; but at that date the planting of these with grasses was commenced. The grasses planted by the conservators of the dunes are principally the helm-grass (*Ammophila arenaria*) and *Triticum junceum*. The roots of these grasses run in all directions, and mat the surface of the sand, thus preventing its dispersal by wind. It is, to the English engineer, an extraordinary sight to see the face of a threatened sand-dune defended by methods so apparently elementary. Where there is a tendency to erosion of the dunes, the practice in Holland is to plant small tufts of helm grass spaced about 12 inches apart. In an old sand-dune the grass will not germinate unless new-blown sand is on the surface; but where the roots of these grasses penetrate, the system is wonderfully effective. On the island of Schouwen the Author saw a sand-dune (Fig 2, Plate 1) threatened with undermining, for the protection of which the engineer depended largely on wisps of grass planted at intervals. In addition to binding the slopes by vegetation, groynes are also frequently adopted as a means of checking erosion. On the less exposed foreshores these groynes are formed of fascine-work protected with basalt blocks, longitudinal rows of short piles being driven to hold the mass together. Groynes on the de Muralt system, described later, are being substituted in some cases for the older type of groyne. As a rule, these groynes are carried vertically but little above the line of the shore. The present condition of the Dutch coast indicates that the dune barrier is now, for practical purposes, almost everywhere stationary. At a point immediately east of the Osse breakwater in the island of Schouwen, the Author was fortunate enough to find an instance in which a work such as this had actually re-established

an ancient line of sand-dunes, which had been recently denuded by natural means. In the North Sea islands (Texel, etc.), which are of vital importance to the defence of the northern Dutch coast-line, and in the province of North Holland, the maintenance of the dunes is in the hands of the Government. In South Holland and Zeeland this duty is performed, in some cases by polder-boards, sometimes by the province, and sometimes by private landowners. From the thirteenth century onwards small polder-boards have been amalgamated to form larger corporations, but all the polder-boards are under official jurisdiction.

The dune barrier, on which the existence of the kingdom of Holland depends, is in many places 1 to 3 miles wide, and in some places it attains a height of 130 feet.

The protection of the dunes on the land side is as important as the maintenance of their seaward slopes. The gradient of the sand on the landward side varies considerably, owing to the fact that sand is blown over the crest of the dunes and falls on their lee side, drifting to a flat slope. In order to arrest this landward travel, the Dutch practice, for many generations, has been to plant trees—mostly fir, beech, and oak—at the foot of the dunes, and to plant shrubs which grow in sandy soil on the slopes. The effect of these trees and shrubs is twofold. The roots of the trees hold the sand together and bind the slope so that it becomes less mobile; and sand blown over the crest of the dunes is caught by the trees and precipitated, being thus saved to the dune.

There is distinct evidence that in some cases an entire range of sand-dunes has, within historic times, shifted bodily landward. In one instance a Roman village, thus buried in sand, was slowly unearthed.

The fineness of dune sand is shown by the following figures:—

Residue on a sieve of 2,500 meshes per square inch	. .	1 per cent.
" " " 5,800 " " " "	. .	4½ "

The origin of these sand-dunes is obviously due to the former existence of large areas of sandbank which, probably owing to some elevation of the land, became exposed to wind-action. The prevailing west and south-west winds caused the throwing-up of sand-hills, just as sand-ripples are formed on the seashore. This effect is the result of minute differences in size and weight of the grains of sand. As may be seen in sand-ripples on any shore, the windward slope is much steeper than the leeward slope: in the case of the sand-dunes the wind sets up a digging action and distributes the spoil at a flat inclination on the lee side.

METHODS OF ARTIFICIAL PROTECTION.

The coastal barriers of Holland consist of massive dams running parallel with the shore-line, in addition to which groynes (*golfsbrekers*), 150 to 200 yards apart and about 500 feet long, are often laid approximately at right angles to the dam, for protection. The seaward slopes of the dams are sometimes built of flat slabs of stone, though in recent years these slabs have been very commonly replaced by basalt (mostly obtained from the Andernach district), which is broken into lengths of about 15 inches and carefully pitched on the exposed portions of the dams. Such work is obviously very expensive; on the Osse breakwater (Figs. 6, Plate 1) in the island of Schouwen, one estimate for basalt pitching amounted to £1,750, or £3 12s. per lineal foot.

Above the exposed level the Dutch employ a method of protection which, so far as the Author is aware, has not been used in any other part of the world. About a handful of straw or rushes is bound into a small sheaf; such sheaves are then laid on the slope at right-angles to the dam, and a wisp of straw is laid over each sheaf; by means of a special tool the Dutch workman presses the straw-band into the clay and fixes the thatching in position. This makes a covering about 1 inch thick, which has to be renewed each year. A labourer can lay 70 square yards of straw thatching in a working-day, between 7 a.m. and 4.30 p.m., for which he is paid $\frac{1}{2}$ d. per square yard, the total cost of the thatching amounting to 2.1d. per square yard. Sometimes brick tiling 2 inches thick is laid on the top of the straw; above the tiling comes random broken brick, and on the surface is basalt pitching 15 or 16 inches thick, packed with broken brick.

Basalt has been in use for about 45 years; before that a limestone rock from Dornik was principally used.

Taking a typical case, the cost of a covering such as that described is:—

	s.	d.	
Clay	3	6	per square yard.
Straw cover	0	2	" " "
Brick tiling	1	4 $\frac{1}{2}$	" " "
Basalt pitching	9	9	" " "
Labour.	1	1 $\frac{1}{2}$	" " "
Total	15	11	" " "

In the Essex marshes low-lying clay lands have to be defended, and the practice of sea-walling, as carried on there, differs materially

from that in vogue on the sandy foreshores of Holland. Fig. 7, Plate 1, shows a typical cross section of the sea-walls in the Thames estuary, on the Essex side, under the supervision of the Author. It will be observed that, as compared with most of the Dutch sea-walls (e.g., Fig. 8, Plate 1), the English section is steeper and has less seaward prolongation. The Dutch practice is to carry the crest of the wall higher than in England; and the crests of the great defensive dams in Holland are built up to a level corresponding with 20·7 feet above Ordnance datum, whereas the maximum level on the Essex coast is 18 feet above that datum.

These Essex sea-walls are laid on the seaward side with a slope of about 2 to 1, and are usually covered with about 8 inches of chalk next to the clay. The chalk is covered with 12 inches of Kentish ragstone pitching, generally carried to within 3 or 4 feet of the top of the wall; and the toe of the slope is sometimes protected by a row of elm piles, about 5 or 6 feet long, driven by hand. Random stone pitching is also put in at the toe of the wall below low water. The average cost of protecting Essex walls in this manner is about 4s. 6d. per square yard, irrespective of the piling, random stone, and clay bed.

An extraordinary phenomenon occurs in connection with submarine slopes on many parts of the Dutch coast, namely, the prevalence of what are called "falls." Fig. 3, Plate 1, illustrates such a fall. The Author was fortunate enough to find at Westkapelle an official (Mr. Dekker) who had actually seen five falls take place near Veere, Walcheren. He described their effect as like that of an earthquake on the sea-bed, great local commotion being caused, and waves 10 or 12 feet high being developed. At some of the river-mouths there are deep channels close inshore, which are 100 to 135 feet, and sometimes even 165 to 200 feet, in depth. The sandy flats under water are very mobile; and at low water the earth-pressure sometimes sets this half-fluid mass in motion. Large areas of the flat, together with the clay on which it rests, then burst away, and an underwater avalanche is the result. Areas 20 acres in extent and 80 to 100 feet in depth have thus given way suddenly; and in the years 1887-96, 106 of these falls occurred.

The protection of the bed of the sea to avert such calamities is effected by various kinds of mattress-work (*zinkstukken*).¹ Mattresses of timber and fascines are floated to the site to be protected, sunk under a load of stone, and subsequently covered with stone pitching.

¹ Minutes of Proceedings Inst. C.E., vol. clxiv, p. 378.

These rafts sometimes measure as much as 25 yards by 300 yards ; and the great art in depositing them is to secure their settlement without dislocation. With this object, the raft, when moored over the place where it is to be deposited, is surrounded by a number of boats laden with masses of rough stone. At a given signal blocks of stone are thrown into the centre of the raft, which sags with their weight and eventually takes the ground, first at the centre and the top end. It settles until it has a fair bearing over the bed of the sea to be protected, and is then covered with riprap. Concrete mattresses are now much in use, and the method is one of wide application in dealing with sandy bars at harbour-entrances. The object of sinking these mattresses is to seal over dangerous areas and avert the contingency of a fall of the sea-bed.

THE DE MURALT SYSTEM OF PROTECTION.

This system is now much in vogue on the Dutch coast, defences costing more than £50,000 having been carried out within the last few years. Mr. de Muralt is a polder-engineer for the island of Schouwen, where the problem of defence is acute.

Concrete mattresses on the de Muralt system have been adopted instead of the old type of fascine mattress, which requires frequent renewal. A series of concrete slabs, about 1 yard square and 5 inches thick, are laid on a flat which is exposed when the tide recedes. At the corners of each slab are iron attachments, by means of which adjacent slabs are linked together, forming a mattress which sometimes has an area of as much as 450 square yards. A flat pontoon is floated over the slabs, and at low water each slab is fastened by a chain to the underside of this pontoon, which rests on timber supports. The upper ends of the cables are attached to windlasses, which are distributed evenly in chess-board fashion, and are geared together, by means of shafting and bevel-gearing, so that after the pontoon has floated up with its load at high water and been towed to the intended site, the whole mattress can be lowered uniformly on to the sea-bed. The de Muralt concrete mattress is a highly efficient arrangement, and it is stated to be cheaper than the old system. Mattress-work may be laid either on a sloping or a level bed.

Above low-water mark, Mr. de Muralt adopts two methods for protecting sea slopes. One of these is employed in exposed situations and for facing dams on which heavy seas are experienced ; the other is suitable for less exposed positions.

In the lighter system, concrete tiles of two types are employed (Figs. 4, Plate 1), which are 16 inches by 16 inches, and $2\frac{1}{4}$ inches thick. They are arranged with rebates at the edges, so that each pair of alternate blocks holds the intermediate block in position. Alternate blocks are keyed firmly to the slope by a reinforced-concrete pin about 18 inches long, which is driven home in the slope and secures the whole sheathing. On a slope thus defended slight settlement may take place without detriment. In several spots the Author found that this had happened where the light type of facing had been used, and that the facing had adjusted itself to each settlement without cracking or serious distortion.

The heavier type of defence is illustrated in Figs. 5, Plate 1. A number of concrete slabs, having a stepped surface and placed at intervals of about 10 inches, both longitudinally and transversely, on the foreshore or bank to be defended, are constructed in situ. Between these slabs the bank is trenched to a depth varying according to circumstances. In the trenches steel reinforcement is placed, and mass concrete is then filled in. The beams thus built up form a casement between the slabs, and are carried above and over the edges of the latter, so that in fact they form a framework, from which the slabs cannot escape. The slabs themselves are reinforced to prevent cracking from earth-pressure beneath during frost. They are free to expand and contract, which is an advantage not obtained, so far as the Author is aware, with any other system. Asphalt joints are provided in the beams, at intervals of about 4 yards, to allow for expansion and contraction of these members.

PROTECTIVE WORKS.

Travelling northward from Flushing, the principal dams on the Dutch coast are the following:—

Island of Walcheren: Westkapelle Dam.—This dam runs into sand-dunes at its southern and north-eastern extremities. Its length is rather more than 3 miles. The bursting of this dam would flood the entire island up to the town of Middelburg, $8\frac{1}{2}$ miles distant. A fall recently took place under water opposite the dam, and a portion of its stonework slipped about 12 feet (Fig. 3, Plate 1). Most of the dam is pitched with basalt, but in some parts Dornik limestone is used.

On the coast of the island of Walcheren there are also considerable lengths of sea-frontage protected by the de Muralt system.

Island of Schouwen.—Apart from subsidiary works, two sea-defence works are of special interest. These are the Osse break-water (Figs. 6, Plate 1), which was executed on the de Muralt system at a cost of £1 11s. 5d. per lineal foot, the corresponding estimate for basalt pitching being £3 12s. per lineal foot. The Langendijk (Fig. 8, Plate 1) lies east and west on the north coast of the island. It is protected partly by "stockade work" consisting of isolated piles which are driven in rows and act as wave-breakers.

Much difference of opinion exists among Dutch engineers as to the policy of adopting stockade work in the design of exposed dams. The argument on one side is that the stockade causes much broken water and consequent local disturbance of stone pitching. The other view is that its effect is to break the blow of the waves without undue surface disturbance, and that it is thus suitable for exposed localities.

On this dam the stockade work was originally in timber, but it has been replaced by reinforced-concrete piles, in two tiers 3 feet apart. The piles are round in section and 7 inches in diameter, and are pitched about 2 feet apart. Along the crest of the dam a de Muralt concrete coaming is built, 4 feet 3 inches high by 1 foot 3 inches wide. This coaming cost 5s. 4½d. per lineal foot.

Renesse.—Adjoining the lighthouse at Renesse is an interesting work constructed to prevent the undercutting of the dunes (Fig. 9, Plate 1). The toe of the dune was eroded, and there was considerable risk of a breach in the line of dune defences, which would have been disastrous. An artificial sandbank 80 feet wide was thrown up in advance of the foot of the dunes and sheathed with a covering of concrete slabs on the de Muralt system. The length of this work is 3,930 feet, of which 1,635 feet lies to the west and 2,295 feet to the east of the lighthouse.

The cost of the concrete slabbing was 4s. 11½d. per square yard, and it was estimated that had the same plan been followed, but basalt pitching on a clay foundation been used, the work would have cost 13s. 11d. per square yard.

The drift of the sand has caused a deposit between the new work and the foot of the dunes, and the area of erosion is thus now almost obliterated. Above the concrete slabbing the face of the dune is planted with tufts of helm grass spaced about 12 inches apart.

Hondsbosse Dam (Figs. 10, 10a, and 10b, Plate 2).—This work is situated in the province of North Holland, on the coast of the North Sea, and is approximately 3,900 feet long, running nearly north and south. It was reconstructed in 1874, and it was then deemed advisable to cut off the stockade-piles flush with the surface of the basalt pitching, which had become loosened by the straining

action that took place at the foot of the piles during heavy seas. The dam was originally pitched with Dornik stone, nearly all of which has now been replaced by basalt.

The dam is under the control of a polder-committee. It runs at its north end into the Petten dam, and this line of defences is one of the most important sea-works of that character in Holland, a breach 3 miles long, due to a gap in the sand-dunes, being closed thereby. The cross section of the seaward face shows the following construction :—

Basalt pitching, packed with broken brick . .	1 foot 5½ inches.
Broken brick	6 to 8 inches.
Brick tiling	2 inches.
Straw	1 inch.
Clay	3 feet 3½ inches.

The cost of the reconstruction amounted to 15s. 11d. per square yard.

Petten Dam (Figs. 11, 11a, and 11b, Plate 2).—This dam is about 1,100 yards long. At the junction with the sand-dunes, at its north end, there is a basalt spur groyne 150 feet long. In the angle sand has accumulated to a depth of 3 feet. Stockade-piles still remain, and have not been cut off flush, as in the case of the Hondsbosse dam.

The Petten dam is under direct Government control, and is not administered by a polder-committee. The cost of maintenance is £1,670 per annum.

As showing the vast importance of the defences provided by the Hondsbosse and Petten dams, it may be mentioned that two safety-dams have been constructed inside them. The lines of this triple system of defences are called respectively by the Dutch equivalents for the English words, “waker,” “dreamer,” and “sleeper.”

Helder Dam (Figs. 12 and 12a, Plate 2).—This is basalt-pitched and surrounds the town of Nieuwediep, which lies immediately to the west of the town of Helder. There is no second line of defence behind this dam, and it is one of the most remarkable in Holland, in that its underwater slope runs down with a uniform gradient of 1 in 1 to a depth of more than 130 feet.

Prinz Hendrik Dam, Texel.—This is on the east coast and extends in a north-easterly direction from the sand-dunes in the south of the island, for a distance of about 2½ miles. It is joined up with the harbour-works at Oudeschild. The most exposed portion is the south end, where the coast-line projects in a knuckle which is fully exposed to extreme gales from the north-east. Here the defence at low-water level consists of random blockwork placed inside

hurdling. The upper slope is 1 in 1, and the lower slope 1 in 8. It is noteworthy that the upper slope is slightly convex in section.

Zonnemaire Dam (Fig. 13, Plate 2).—This dam lies east of Brouwershaven, in the province of Zeeland. Its exposed face is protected with reinforced-concrete slabs, and along the crest is a reinforced-concrete coaming, about 3 feet high, which obviates the necessity of widening the dam at the back and thus encroaching on the cart-road at its foot.

ZUIDER ZEE RECLAMATION PROJECTS.

The Zuider Zee (*Fig. 14*) now covers an area of more than $1\frac{1}{2}$ million acres. Its depth increases from south to north and averages 13 to 16 feet: in the vicinity of the islands depressions exist. The average range of tide is only about 10 or 12 inches, and at the south end it is barely perceptible; north-westerly gales, however, sometimes raise the water-level 7 or 8 feet.

In view of the many discussions which have taken place as to the possibility of reclaiming the Zuider Zee, a short statement of its geographical conditions and a summary of modern proposals for its partial reclamation may not be out of place. The conversion of this immense area of shallow sea into dry land, and the consequent acquisition by Holland of a new province conquered by the arts of peace, is the dream of every Dutch patriot. Obviously, the expenditure of a vast capital sum on the reclamation of a tract of barren sand would be undesirable. Large areas of the bed of the Zuider Zee consist of fertile mud, which could be converted into valuable agricultural land; other areas, however, are almost exclusively sandy, and their reclamation would serve little purpose.

If a line from Veir, a little north of Enkhuizen, is laid out to the island of Urk, and then to the mouth of the IJssel, an area of about 500,000 acres is included. Four-fifths of this area is covered with rich soil 4 or 5 feet thick; the remainder is sandy. Laying out a second line from Veir to the coast of Friesland, near Stavoren, the area included is about 200,000 acres. The bottom of this area is for the most part of thick mud, the remainder being sandy. In the Zuider Zee the rivers of western Germany find their outlet.

The first serious studies in connection with the draining of the Zuider Zee were made by Van Diggelen in 1849. His project was of vast proportions, as he proposed the reclamation of the entire area, good and bad, and the control of the various rivers flowing into it. Briefly, he proposed a coastal canal, running from Hoorn to the north-west of Friesland, which was

18 CAREY ON WINNING COASTAL LANDS IN HOLLAND. [Minutes of
to have a width of 2 to 12 kilometres ($1\frac{1}{2}$ to $7\frac{1}{2}$ miles) and
was intended to be protected by two lines of dams. Two escape-
canals into the North Sea were also to be cut, one from Hoorn

Fig. 14.



MAP OF THE ZUIDER ZEE, SHOWING RECLAMATION-SCHEME PROPOSED BY
THE ROYAL COMMISSION OF 1892.

to Petten, the other discharging at Vlie. Another entirely
independent canal was designed to traverse the provinces of
Overijssel, Drente, and Friesland, collecting the land-drainage of
that area, and discharging into the sea at Pesens, to the west of

the Lauwers Zee. The northern islands of Holland were to be connected by two enormous dams, to protect the reclamation from the north-westerly storms that are so dangerous on that coast. Van Diggelen's estimate for this project was between 27 and 28 million pounds.

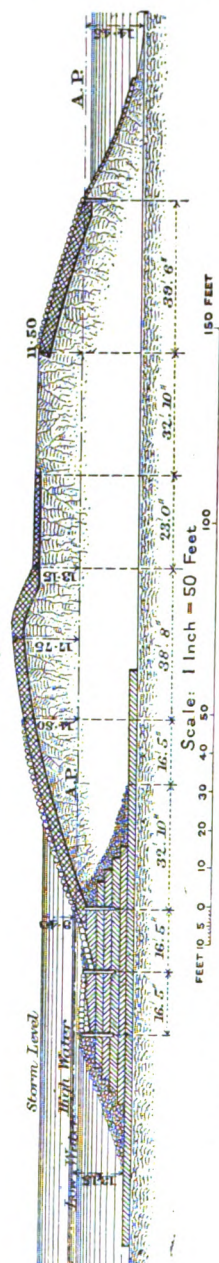
In 1865 the scheme was revived in a modified form. Colonel Beijerinck was deputed to report to the Dutch Government, and in conjunction with Mr. Stieltjes he spent 2 years in examining the bed of the Zee. Ultimately a State Commission was nominated, which presented a favourable report in 1873. A Bill was presented to Parliament in 1877 for embanking and draining the Zuider Zee and for the construction of a navigable canal between Amsterdam and Valhal. The State was to provide the necessary capital (£10,250,000), and the period for the execution of the works was not to exceed 16 years. The Second Chamber of Parliament passed a provisional credit for £600,000 for a complete investigation of the entire project. The question of the intended works was then submitted to the country, with the result that the ministers who had formulated the proposals described were defeated at the polls.

In 1882 an attempt was made to revive the proposals for the reclamation of the Zuider Zee. In 1892 a Royal Commission was appointed, the Chief Engineer being Mr. C. Lely. His proposal was to build a dam from the coast of North Holland via Wieringen to the coast of Friesland, near Piaam, and to drain four polders (Nos. 1, 2, 3, and 4, *Fig. 14*) successively, leaving a central lake, to be called Lake IJssel. The Royal Commission generally approved Mr. Lely's plans. The length of the dam so designed was about 17.6 miles (29,300 metres). The original intended average section of this work is shown in *Fig. 15*. The estimate for its construction was £3,375,000; but to cover insurance of workmen and other charges, this estimate was increased to £3,700,000. Including defensive work and compensation for fishery-rights, etc., the total cost was estimated at £9,900,000. As, however, the construction of a work of such magnitude would occupy two generations, it has been argued that the slow extinction of the fishing-industry in the enclosed area would be automatic, and should not involve State expenditure.

The application of the de Muralt system in the execution of this work was proposed in 1907 by Mr. L. A. Sanders, of Amsterdam.¹ Briefly, his suggestion was to sink reinforced-

¹ L. A. Sanders, "De Toe-Komst van Cement-Ijzeren Putten en Platen op Waterbouw-Kundig Gebied," Amsterdam, 1907,

Fig. 15.



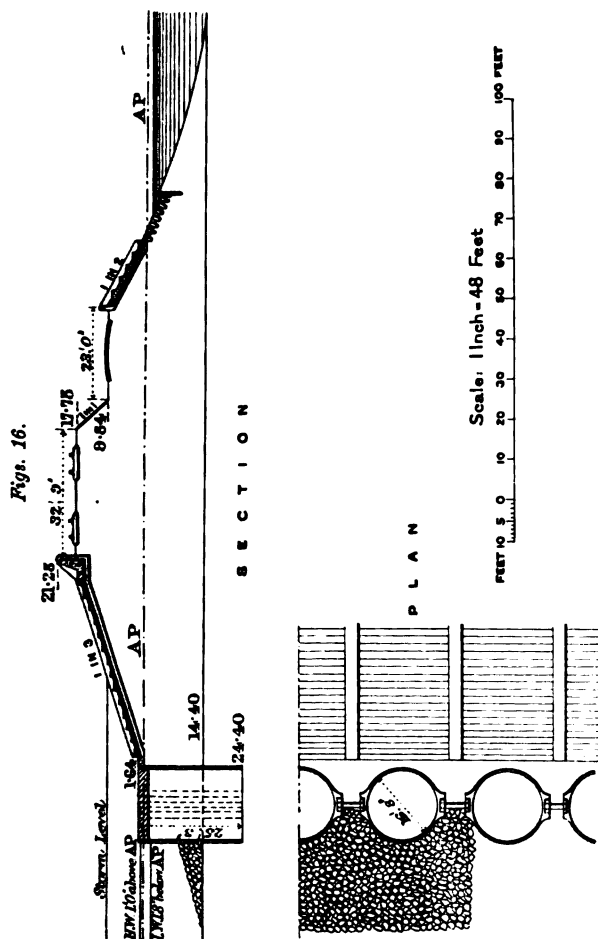
ZUIDER ZEE RECLAMATION: AVERAGE SECTION OF THE DAM PROPOSED BY THE ROYAL COMMISSION OF 1892.

concrete cylinders 16 feet 10 inches in diameter and 26 feet 8 inches high, and to drive reinforced-concrete shields between and connecting these (*Figs. 16*). The cylinders were to be filled with clay and sealed at the top with concrete 2 feet 3 inches thick. This work, carried to the height indicated, would, he estimated, permit of a clay embankment being tipped to the finished level (5·4 metres A.P.). The embankment would be wholly protected on the exposed side, and partially protected on the lee side, on the de Muralt system. He estimated the cost of such a dam at £2,700,000. Mr. Sanders's idea was to construct the isolating dam in its entirety as the first section of the reclamation-work. This plan, he stated, would represent a large saving as compared with the suggestion to make the reclamation of the four polders the first work.

The objections raised against Mr. Sanders's proposal are the risk of settlement of the cylinders, and their liability to be overturned from behind by earth-pressure or scoured from beneath by the sea. The question of the reclamation of the Zuider Zee, however, is one which public opinion in Holland will probably force to a practical issue at an early date.

In conclusion, the Author wishes to express his great obligations for the courteous assistance he received in his inspection of the Dutch coast-line. He desires especially to thank Professor Beekman of The Hague; Messrs. A. T. de Groot, Rijkswaterstaat engineer, Alkmaar; A. van Gelderen, Waterstaat engineer, Mid-

delburg; R. R. L. de Muralt, Polder-engineer, Zierikzee; and Messrs. Schotel and Dekker, in charge respectively of the Helder and Westkapelle dikes.



ZUIDER ZEE RECLAMATION : DAM PROPOSED BY MR. L. A. SANDERS.

The Paper is accompanied by twenty tracings, from which Plates 1 and 2 and the Figures in the text have been prepared.

[DISCUSSION.

Discussion.

The President. The **PRESIDENT**, in moving a vote of thanks to the Author for his Paper, remarked that it should be of especial interest at the present time, having regard to the fact that a Royal Commission on Coast-Erosion was sitting, which had collected a great deal of information about the erosion and accretion of the coast of certain parts of the United Kingdom.

The Author. The **AUTHOR** observed that in passing through the kingdom of Holland two points struck the foreigner very forcibly: one was the care devoted to agriculture, and the other was the simplicity of the expedients adopted to protect the country from inundation, and the success with which those simple expedients were attended. It was obvious that in Holland agriculture was a thriving industry and that the success of farming was due primarily to rigorous economy and business-like methods. The Dutch had solved the problem of preserving a rural population on the land, and, considering that fact, he had ventured to lay before The Institution the results of his recent visit to their country. In Great Britain what might be termed professional farming was rather under an eclipse. Rich men did not go into farming primarily for the sake of profit, and what was wanted was to bring back, very much on the Dutch lines, the small yeoman, tilling his own acres. Any information as to foreign practice that would throw light upon the subject was therefore worthy of discussion. The Author then exhibited a number of lantern-slides with the assistance of Mr. de Muralt, who also explained, by means of models, the systems of construction applied by him to sea-defence works.

Mr. Wheeler. **Mr. W. H. WHEELER** remarked that the work described in the Paper was of very great interest to the civil engineer. Holland owed its existence as an inhabited country entirely to the works of the engineer; and it continued to exist only by reason of the fact that a large body of engineers kept careful watch over its sea-defences. Holland was a remarkable place—the last land to be made in the world. It owed its origin to the debris brought down by the four great rivers discharging their waters into that part of the North Sea, from Switzerland, France, Belgium, and Germany, some of the material having been transported by the water for a distance of

about 500 miles. That material, brought down by the rivers and deposited in the North Sea, formed a delta stretching out from the original coast-line for 130 miles, with a width of 60 miles. When the land was raised to the condition of a marsh, the first reclamation was made by Nature. The sand on the wide stretch of shore left dry by the tide was blown shoreward, and, being arrested by the marshlands above the level of the ordinary tide, it gradually accumulated and formed banks or dunes, thus creating the first outer barrier against the encroachments of the sea. Those dunes were a remarkable phenomenon. They commenced on the coast of France where the chalk cliffs terminated, a little beyond Calais, and continued in an almost unbroken line along the coasts of Belgium and Holland, around by Texel, and along the coasts of Germany, Denmark, Jutland, and Schleswig-Holstein, a distance of more than 1,000 miles. Over the greater part of that distance they ranged from 30 to 50 feet in height, but in some places they attained a height of 200 feet above sea-level and a width of $2\frac{1}{2}$ miles. The dunes formed the main protection against the inroads of the sea, but there were many weak places requiring the aid of the engineer to strengthen and maintain them. The reclamation of that tract of marsh and bog-land was first undertaken about 2,000 years ago. When the Roman engineers first set to work it was described as an amphibious territory over which the sea had no defined limits, the rivers no constant beds, and the earth no solidity. The few inhabitants who lived on the higher ground or islands, and who obtained a precarious existence by fishing and fowling, were described as being like navigators when the tide rose, and like shipwrecked mariners when it fell. The Roman engineers commenced the making of Holland by closing the gaps in the dunes, and embanking and controlling the rivers; and they succeeded in making a large tract available for the cultivation of corn, which was used for feeding the large armies kept in that part of Europe. That reclaimed land formed the nucleus of what in time became the kingdom of Holland, covering 13,000 square miles of the richest agricultural land in Europe, and bearing a population of $4\frac{1}{2}$ millions of industrious inhabitants. The land gained then and subsequently had been maintained only by long and continuous fighting, with some defeats and many victories, and to-day it was held only by the unremitting attention of the large staff of engineers who had the country under their charge. At one time, owing to gales and high tides, the sea burst through the protecting dunes, destroying towns and villages. At another time the rivers burst the barriers imposed upon them and spread their waters over the fertile lands, destroying the

Mr. Wheeler. inhabitants by thousands. There was scarcely a foothold in the lower part of Holland that had not been submerged within historic times. In one of those inundations the floods poured over 20,000 acres of land and swallowed up thirty-nine villages. On another occasion, owing to a great storm, the North Sea was raised to an unprecedented height, the water was driven through the protecting banks, and more than 1,000,000 acres of land were engulfed, with all the villages and churches and homesteads, destroying a population of 780,000 persons. The land thus destroyed was never reclaimed, and now formed the Zuider Zee. Many schemes had been brought forward for the reclamation of that portion of the country, the more recent of which were referred to in the Paper. Two centuries after that inundation, during an extraordinary tide, a breach occurred in the banks of one of the rivers that passed through the country, and upwards of 100,000 acres of rich agricultural land, with thirty-five villages, was converted into a swamp, and 100,000 persons lost their lives. About a century later, in 1570, the sea broke through the banks protecting Friesland, inundating the country and destroying 400,000 persons, with houses and churches. This district after that remained for many years in a lost condition, as, owing to the great loss of life, there was such a scarcity of labour that men could not be found to restore the work. In more recent times, owing to the better organization of the means of protection, calamities of such large proportions had been prevented; but the country was still held secure only by the most unremitting attention to sea- and river-defences. There were 1,500 miles of coast-dunes and 500 miles of river-banks that had to be kept under constant supervision, and were only maintained at a cost of half a million sterling; and about 9 millions sterling had been spent within the last few years in improving the works. The Author gave a very interesting and instructive description of the foreshore and sea-defence and other beneficial works for the protection of the country, but those constituted only a part of the works that had to be maintained. A large part of Holland—in fact, the polder-land, which constituted more than one-third of the total area—was several feet below sea-level, and was maintained only by constant pumping. The machinery used for that purpose first came into operation about 500 years ago, and the same type was still very largely in use. It consisted of scoop-wheels driven by windmills. The wheels were similar in construction to the water-wheel used for mills, but with the difference that in the one case the water turned the wheel, and in the other the wheel drove the water, forcing it up an inclined plane. The wheels generally raise the water 8 to 10 feet, and when the lift much exceeded that

height the wheels were placed in steps one above the other. Wheels Mr. Wheeler. up to 50 feet in diameter were to be found. The windmills had been almost entirely replaced by steam-engines, and the scoop-wheels were gradually giving place to centrifugal pumps; but up to 60 years ago the windmills continued in use as the only mechanical means of raising the water. The use of steam for pumping was introduced into Holland about 60 years ago, for the reclamation of Lake Haarlem, a polder, or lake, covering an area of 57,000 acres, the bottom of which was 11 to 16 feet below sea-level. Before the land could be prepared for reclamation, 900 million tons of water had to be pumped to a height of 16 feet. At first it was proposed to accomplish that by scoop-wheels driven by wind-power working with a double lift, and it was estimated that it would require one hundred and fourteen windmills; but ultimately the design of two English engineers was accepted for the erection of steam-pumps somewhat similar to those used for raising water from the Cornish mines. The engines and pumps were of an entirely novel type. Three pumping-stations were built at different parts of the lake, each of which was fitted with engines of 350 HP. The engines were beam-engines, worked by two cylinders, one placed inside the other. The cylinders were 12 feet and 7 feet in diameter respectively, and 10 feet high—the largest that had ever been used up to that time. The pumps, eleven in number, were arranged in a circle around the steam-cylinder, and connected to it by beams 30 tons in weight. As the piston rose and fell, it actuated the beams attached to the pumps. The engine made ten strokes per minute, the quantity of water lifted being about 6 tons per stroke, so that, altogether, about 660 tons of water was raised every minute. Even with those powerful pumps it took nearly $3\frac{1}{2}$ years to pump the water out of the lake, so as to allow the work of reclamation to be commenced. Those engines, erected by English engineers, remained in use for about 40 years, and within the last 7 years or so they had been superseded by centrifugal pumps, also supplied by English engineers. A full description of those pumps and other works in Holland would be found in a book¹ in the library of The Institution. The Author drew a comparison between the works for the protection of Holland and those used in the Essex marshes, and gave a section of embankment used in Essex. It seemed, however, that the fenland on the east coast of England afforded a better basis of comparison, as it

¹ W. H. Wheeler, "The Drainage of Fens and Low Lands by Gravitation and Steam Power." London, 1888.

Mr. Wheeler. comprised a larger area of land similar to the polders, which had to be maintained in a fit state for cultivation by pumping, scoop-wheels similar to those used in Holland being largely employed for the purpose. The Author also remarked that to the English engineer it was an extraordinary sight to see the face of the sand-dunes defended by means so apparently elementary as the protection afforded by the binding action of the grasses that grow in them. But the same method was in use in the sand-dunes that protected the fens of Lincolnshire, extending from the Humber to the Wash (a distance of 30 miles), and also on the Norfolk coast along the Broads, and on the west coast. The grass most commonly to be found in those dunes was known in England as marram-grass, or botanically as *Arundo arenaria*. There were several other grasses and small shrubs that flourished and assisted in the protection; and it was the great aim of those who had charge of the maintenance of the banks to see that no injury was done to the grasses. There were many more interesting engineering features that might be mentioned, but he thought he had said enough to justify the statement that Holland was essentially an interesting country from an engineering point of view.

Mr. Grantham. Mr. R. F. GRANTHAM observed that in times gone by Great Britain had learned a good deal from the Dutch, and possibly something more might be learned from the system they were adopting now. In comparing the Essex defences with those of Holland, the Author, he thought, did not choose examples which were quite comparable; the Lincolnshire marshes would, as Mr. Wheeler had suggested, afford a better comparison. As to the height of the sea-defences, about 30 years ago Mr. Grantham fixed the height for the walls around Canvey Island in Essex at 18 feet above Ordnance datum; but in reclaiming an area of 340 acres in Lincolnshire this year the top of the bank had been fixed at 20 feet above Ordnance datum, and along the Lincolnshire coast the walls were rather more than 20 feet high, so that they were almost the same as the defences in Holland. He had records which showed that the Dutch were at work rather earlier than the Author mentioned. In 1616 a Dutchman undertook to repair the walls of Canvey Island, provided he was given one-third of the island. That was agreed to, and the allotted lands were known to-day as "Third-Acre Lands." In 1620 Sir Hugh Myddelton employed Dutchmen to assist him in enclosing the Brading harbour in the Isle of Wight, and he reclaimed that harbour and shut out the tide; but a few years later the tide broke in again, and the area remained under water until 1880, when the sea was again shut out. In 1626 Dutchmen were employed in

Lincolnshire. The Author mentioned various names for reclaimed lands, such as "polders" in Holland, and "levels" in Essex, but in Lincolnshire they were called "marshes"—not "fens," there being a distinction between marshes and fens in Lincolnshire. Marshes were formed by the silt brought up by the sea, while fens were formed by deposits of the rivers in ancient times, and by decayed vegetable matter. With reference to pumping in Holland and in England, scoop-wheels were still in existence, both in Lincolnshire and in Norfolk. In Norfolk there were also wind-mills still in existence, but lately the practice had been to put down auxiliary portable engines and pumps, so that when there was not sufficient wind to lift the water out of the drains, the engines could be set to work. It was found that the marshes could be drained very effectively in that way. The Author stated that the Commissioners appointed in England need not be interested locally in the particular district over which they had control. Commissioners could be appointed if they lived in the county or had property in the county; but as a matter of practice it would be found generally that Commissioners had interests in the drainage-area which they controlled.

The AUTHOR observed that that was not his experience.

The Author.

Mr. GRANTHAM thought that possibly the Author's experience had been somewhat limited.

Mr. Grantham.

A MEMBER, speaking as a small landowner in Essex, said he knew it was not the case, although it certainly ought to be so.

A Member.

Mr. GRANTHAM said that, according to his experience, the Commissioners of Sewers could be appointed, provided they were interested in the county, and they generally possessed, or were agents for, land in the district which they controlled. He understood from Mr. Wheeler that the helm-grass mentioned by the Author was the marram-grass to be found in Lincolnshire and Norfolk, and also on the banks of the Mersey by Blundell Sands. With regard to sand-dunes wearing away, they did so in Norfolk, where the dunes protecting the coast were partly artificial and partly blown up by the wind. A single view of the sand-dunes and the shore there would not convince anyone that there was encroachment of the sea at all; but the sea was really encroaching. Sand-dunes were gradually blown back, and were undermined at the foot when there was a very high tide; in that way there occurred a serious loss of land, as was shown by the fact that the remains of Eccles church, which used to stand inside the sand-dunes, were now on the shore, the sand-dunes having been actually washed or blown back. He could not say what distance

Mr. Grantham.

Mr. Grantham. the recession had been. With regard to Fig. 7, he had found a 2-to-1 slope generally on back walls up creeks; but in the Thames and Medway on the front slopes, where there was stone pitching, the stone was laid at a flatter slope, and he thought it was better that it should be. A very good plan, however, which he had adopted was to make the lower part of the slope 3 or even 4 to 1, and to turn up the end at the top. That prevented waves from running right on to the earth top of the wall and washing it out. Those remarks applied to Essex; in Lincolnshire it was almost universal practice to lay the front slope at 5 to 1, and many of the banks in the Wash were so laid, and pitched for a few feet up the slope with stone or chalk. In the enclosure he had made this year the front slope was laid at 5 to 1, and so far no pitching had been put on at all, there being so little tide that he did not think any stone facing would be required. Dependence was placed entirely on the grass bank. During the winter he had seen no signs that any chalk or stone would be required. He was indebted to the Author for an opportunity of seeing the lighter section of Mr. de Mural's system, and he formed a favourable opinion of it at the time. It was in rather a sheltered position. All down the Thames there were acres of stone facing the slopes of the walls, and the great difficulty was the cost. The Author showed that the Dutch had been at a great disadvantage, because they had had to use basalt, and it was mentioned that the cost of that basalt pitching with the straw cover, labour, etc., came out at 15s. 11d. per square yard. The Author also said that the cost of stone pitching in Essex was 4s. 6d. per square yard. Mr. Grantham had generally put it down at something under 5s., and in England, of course, stone, chalk, slag, and even large chalk flints, could be obtained very easily at a comparatively small cost; so that he did not think the cost of pitching the face of a wall in this country would be more than 5s. per square yard. He had seen some walls very well made with chalk flints. It was a question whether the de Mural system in long slopes—say, 50 feet long—could supersede the system employed in England, on account of the cost of the former. With regard to the reclamation of the Zuider Zee, it seemed to be a gigantic undertaking, but the rise of the tide, according to the Author, was only 10 or 11 inches, and there would not be the same difficulty in that case as there would be with a greater range of tide.

Mr. Fowler. Mr. A. F. FOWLER remarked that he had had considerable experience lately in Lancashire, and the difference between reclamation of coastal land in Holland and reclamation of land in England seemed to him to be that in the one case it was a national necessity,

and therefore the nation was behind to pay for it, and in the other Mr. Fowler. case it was purely a commercial question, and land was reclaimed only where it could be done cheaply. Therefore the remarks made by Mr. Grantham applied, he thought, to the de Muralt system, the cost of which in many cases would absolutely prevent enterprise in the reclamation of land. The reclamation embankments on the south side of the Ribble were constructed, in the upper half, to a slope of 2 to 1, and in the lower half to a slope of 5 to 1, the toe being protected by a belt of stone pitching 12 inches thick by 9 feet wide. Outside of that, some few yards distant, a stockade was made. A stockade was necessary in the Ribble for the maintenance of the embankment, because it was found that buoys drifted from their moorings in the channel, boats got adrift, and other heavy floating substances, such as logs of timber, seriously damaged the face of the embankment if a stockade was not present. The Author said that in Holland there was a difference of opinion as to the value of stockades, and he mentioned one place where a space of 2 feet was left between the various members of the stockade. That stockade would not seem to constitute an effective barrier to the sea, on account of the width of the space; and moreover it would cause very undesirable concentration of wave-action on the toe of the embankment. Dealing with another question of great interest to many harbour-authorities in England, the Author referred to the control of sea-defences in the low-lying districts of England. Presumably he referred only to the east coast between the Thames and the Humber, but lands reclaimed from the sea in some other parts of the kingdom, such as the estuaries of the Tees, the Ribble, and the Dee, were controlled by harbour-authorities under clauses embodied in Acts of Parliament. In the case of the Ribble a large portion of the lands gained by reclamation were vested in the Corporation of Preston, which was the harbour-authority, but were subject to the right of pre-emption by the owners of ancient enclosed lands adjoining, and subject to the rights of the Duchy of Lancaster. Not much value appeared to be attached to those rights by the duchy, as they had sacrificed very large areas for merely nominal sums. Other portions of the estuary were vested in the land-owners, subject to the payment to the Corporation of Preston of an annual rent-charge agreed upon by arbitration, again subject to the duchy rights where they had not been sold. Once the land was reclaimed, the owner had still charge and unrestricted control. The lands on the Ribble estuary were actual reclamations from the coast-line, and not merely lands subject to flooding. There was,

Mr. Fowler. however, behind them a tract of ancient enclosed land extending over about 38,000 acres, the subsoil water-level of which was still kept down to some extent by steam-pumps, the outer lands being drained by tidal-lock culverts in the ordinary way. With regard to the question of authority over reclaimed land, it was highly desirable from a navigation point of view that there should be some real Governmental control. It had been mentioned that in Holland the agricultural interest was very well looked after. From a general knowledge of Holland and the rivers, such as the Maas, it was evident that the navigation had been looked after in no less a degree, and in the rivers reclamation had been made with due consideration for the navigation. It was indeed probable that the latter had benefited considerably by the consequent regulation of the high-water line. In the earlier history of the Ribble and the Dee large areas of land had been reclaimed with the sole object of obtaining money for the land; and anyone examining the charts of those rivers would recognize how much the flood-tide had been obstructed at the upper part of the estuary, where all the possible obstructions should be removed. The nominal Conservator of the Ribble was the Duchy of Lancaster, but matters affecting the public or navigation-rights were usually referred to the Board of Trade. That Department, in the opinion of some harbour-authorities, was hard to move, particularly when the works undertaken were above high-water mark of ordinary tides. With reference to grass-planting, the sand-dunes between Formby Point and Southport, a distance of about 7 miles, had advanced during the last 20 years a maximum distance of about 75 yards, and that advance, if not entirely due to grass-planting, had been very considerably assisted by it. The grass was of the same nature as the helm-grass referred to by the Author, and its manipulation and effect were precisely as stated in the Paper. When one belt of grass had intercepted enough sand, another belt was planted, so that at present there was a $\frac{1}{4}$ -mile belt of sand-hills presented to the eye as a series of terraces. On the Ribble the grass did not thrive so well in the comparatively sheltered foreshore beyond Southport as it did in the exposed sea portion. The reclamation embankments on the Ribble were about 16 feet in height and about 22 feet above Ordnance datum. At a height of 20 feet they would be just the height of the highest known tide.

The Author. The AUTHOR, in reply, observed that the discussion had been largely descriptive, on points collateral with the subject of his Paper. Mr. Wheeler had alluded to the question of the water-level of the polder-lands. The Author had dealt somewhat fully with this point at p. 6, where he had defined the artificial level at which inland

waters throughout Holland were maintained by law, and the reason The Author. for this unusual condition of things. There were, of course, many situations on the East Anglian coast which closely resembled corresponding localities on the Dutch coast; and in Essex there were large tracts of land almost exactly similar to those of Holland. A Dutchman shipwrecked upon the island of Foulness, for instance, might certainly believe himself to be in his own country. With regard to the plantation of the grasses on sand-dunes, although well acquainted with the Norfolk coast, the Author was not aware of any spot where regular planting of marram-grass was carried out in the same systematic manner as in Holland. He had referred more particularly to this subject in a monograph recently published.¹ A law of George II made it a penal offence to cut or injure the marram-grass, but he doubted if protective measures were carried out anywhere on the English coast with the same methodical rigour as was the case in Holland. The grasses mentioned on p. 9 were helm or marram and the rushy wheat-grass (*Triticum junceum*). With regard to Mr. Grantham's remarks, the Author's comparison was specifically between the Dutch and the Essex practice. In the case of the Essex marshes the standard levels were considerably lower than those on the Dutch coast. On the question of the administration of the controlling boards or Level Commissioners, he acted for a number of these bodies, and while in every case one condition of election was the ownership of real property of a given annual value in the county, commissioners seldom actually owned property within the jurisdiction of the Court. These bodies carried out their functions with much economy and efficiency. As to the cost of facing river-walls, to which Mr. Grantham had referred, the Author's practice hitherto had been to pitch the walls with 8 inches of block chalk covered with 12 inches of Kentish ragstone. The cost of such work ranged from 4s. 6d. to 5s. per square yard. Now, however, he was substituting de Muralt slabs of the lighter type where practicable. He had found the cost of this slabbing the same as that of the chalk and ragstone pitching, and it had the advantage of not requiring constant repair. If the slabbing worked slack, all that was required was to drive home the concrete pins. He thought Mr. Fowler had been under misapprehension as to the cost of the Dutch system. In England there was no material difference in first outlay between stone pitching and concrete slabbing, but there was a distinct advantage in the lower expense of maintenance under the latter system.

¹ A. E. Carey, "The Protection of Sea Shores from Erosion." London, 1907.

The Author. With regard to the policy of driving stockades, in Holland stockades consisted of rows of piles driven one behind the other, the rows being 2 or 3 feet apart, and diagonally pitched, the piling itself being continuous. As stated in the Paper, there was a difference of professional opinion as to the balance of advantage obtained by adopting stockade work or by trusting solely to surface pitching or slabbing. Stockades undoubtedly cut the run of the sea, but in so doing they were apt to shake the stone pitching. The Author's own practice was sometimes to drive short elm piles, at extreme low water, to give support to the stone facing of a bank. They prevented the stonework from slipping forward, and did not tend to disintegrate it under the shock of waves.

Correspondence.

Mr. Bell. Mr. W. REID BELL remarked that the difference between the Dutch and the Essex sea-walls, as shown in Figs. 7 and 8, Plate 1, would appear to be due to the different exposures. The flat slopes of the Dutch beaches were exposed to the long fetch across the North Sea from the north and north-west, whereas the Essex shores had a south-easterly exposure with a short fetch. In view of the great fertility of Holland it was interesting to examine the effect of the high water-level in the soil as influencing the growth of vegetation. At first one would be inclined to think that the presence of water so near the surface would cool the soil below the temperature most favourable for growth; but on reflection it was evident that by the time the underground water reached the surface it must be at the mean earth-temperature of the region. The cold of winter, while freezing the surface, evidently could not penetrate deeply, and on the return of spring the subsoil-water must have a marked effect in hastening the reheating of the surface soil. These speculations were made without practical knowledge of the country, but it would be very instructive to irrigation-engineers if the Author could bring forward any information as to the soil-temperatures in Holland at different seasons of the year, and as to what limitations there were to the maturation of cereals, what grasses and what crops were most luxuriant in their growth, and so on. With the regulated saturation of the soil and the hard winters, the Dutch pastures should be very free from insect pests. What had the farmers to contend with in this direction?

With regard to the statement that the percolation in the sand- Mr. Bell. dunes amounted to 64 per cent. of the rainfall, he would ask whether this was an estimate or was based upon observation or experiment. It seemed to him that the fresh water collected in the dunes from the rainfall would assume a lenticular profile in section, and that the lowering of the saturation-surface by pumping might cause a rise in the salt-water level below. The two surfaces (namely, the upper surfaces of the fresh and the salt water) would meet at sea-level, and it would be reasonable to suppose that any fresh-water drainage from the dunes would exude at about tide-level and not below the level of the sea, as might be the case in the drainage of water held in rocks: in the latter case, of course, water might be carried by geological faults to form submarine springs. It was exceedingly interesting to note that in this flat country the maximum rainfall occurred in the sand-dune regions. It had been fairly well established that the determining factor in deposition was the upward deflection of moist air into colder regions; and this was precisely the effect when the rain-winds encountered the steep seaward slopes of the dunes. Undoubtedly the natural fresh-water reservoirs of Holland were due to this cause. In regard to the planting of trees on the leeward side of sand-dunes, he would take exception to the statement that the living trees were of use in catching the sand blown over the crest. His experience was that, unless the seaward face had been thoroughly protected, the leeward trees were quickly buried and destroyed by the sand coming over the crest. It did not appear to be necessary for the land to be rising from the sea in order to expose the sand to wind-action: high tides and onshore winds heaped up sand at high-water mark, and every time the tide receded the foreshore sand dried and was blown inland in clouds, leaving its place to be filled by sand churned up on the banks at sea. Numerous questions were suggested by perusal of the Paper. The Author stated that the flocks followed the tide as it receded, and it would be interesting to know what the pasture on the tidal lands consisted of? Further, when new polders were reclaimed, did the land won become the property of the State or of the Province, and how was it disposed of and at what price? For colonial engineers Holland should have a particular interest, inasmuch as there the inhabitants increased their hard-won land acre by acre, while the Colonies suffered all the evils of too much land. Only human beings could squander their resources as the Colonies had done! The result of too much land had been bad farming and negligent cultivation, while the cost of communication and administration, and the difficulties of education, had been enhanced beyond reason. In

Mr. Bell. grass countries such as the table-land of South Africa, except for the introduction of pests and the destruction and desiccation of arable country by erosion of watercourses following on man's occupation, the actual injury done to the land was not irreparable. In countries such as Tasmania and the east-coast regions of Australia, however, where vast and valuable forests were continually being destroyed by fire for the sake of the stock-breeder, who practically carried on no cultivation, or of the pioneer farmer, who continually hungered for fresh land to "clear," the evil was truly irreparable. The evils of the system of free selection of land were well known, but very little effort to remedy matters could be expected unless a strong political party arose that would devote itself to the one question of the development of the country for the benefit of the country. With regard to the wonderful structures raised in Holland to keep out the sea, the information given in this instructive Paper prompted those who were not acquainted with the country to ask for more. An account of how these dams were first raised above the sea-level and finally closed would be of great value. It was only in the Zuider Zee that the tidal variation was small, and the closure of polder-dams where the tidal range was 5 to 10 or more feet must be an anxious and critical operation. In such a country, too, where land was so scarce and water so plentiful, the average engineer wondered how even the excavation of the dikes and the accumulating reservoirs could furnish enough material for the dams; and he would like to know how it was proposed to obtain material for the great dam to form Lake IJssel. The word dike seems to be derived from the same root as dig. The "diggings" of the Romans in Scotland were for throwing up the walls of fortifications, and in Holland the importance of the digging was the dam, while in Essex and Somersetshire the channel for drainage was the object of the digging. Hence, probably, the different significance of the word in the several localities.

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Mr. V. I. P. DE BLOCQ VAN KUFFELER, of the Waterstaat, felt that the Paper took a broad view of the subject, and the fact that it required some additions showed once more how complicated was the Dutch system of sea-defences, and how difficult it was for a foreigner to grasp it all in a short time. The Author had visited the principal sea-walls, and if he had also seen the sea-wall of the Waard and Groet polder, which was only 3·87 metres (12·7 feet) above A.P. at its crest, with a seaward slope of 1 in 1·7, he would have been less surprised at the difference between Essex and Holland. In comparing dimensions attention should be paid to the difference in the conditions to which the walls were exposed,

e.g., situation with respect to storms, depth of water along the coast, shape of the coast-line, existence of sandbanks or other protection, etc. He supposed that the walls of the Waard and Groet polder were among those with the smallest dimensions in Holland. The height of the crest also depended on circumstances. At Westkapelle it was 7.1 metres (23.3 feet) above datum (A.P.); but there ordinary high-water level was 0.9 metre (2.95 feet) above datum, with a low-water level of 1.5 metre (4.9 feet) below it. The crest-level of the Hondsbosse dam was 7 metres (23 feet), and of the Petten dam 6 metres (19.6 feet) above datum. The crest of the dam projected for the reclamation of the Zuider Zee, however, was to be only 5.2 metres (17.1 feet) above datum. High-water level at Wieringen was only 0.25 metre (0.83 foot) above datum and low-water level 0.62 metre (2.03 feet) below. It was therefore impossible to fix a standard for the height of a wall-crest.

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With regard to the general formation of alluvium in Holland, something might be added to give a clearer description of the state of the soil, and as to the origin of the Zuider Zee, as still related according to the old legend. It was supposed that, towards the end of the diluvial period, flats were formed beyond the diluvial beach, on which flats a chain of dunes arose. This state of things would resemble what was found nowadays in the Baltic. There, in a *Haff*, with its calm water protected by a *Nehrung*, opportunity had occurred for sand to settle—the coarser nearer the outlets to the sea, the finer nearer the land. Thus, seaward there was sand, landward clay, and between the two a mixture of sand and clay. If this raising of the bottom were accompanied by a narrowing of the outlets to the sea, the tidal movement in the *Haff* would diminish, and so would the proportion of salt, through the influx of water from the surrounding higher ground. On the newly-formed soil arose a vigorous vegetation which led to the formation of layers of peat. As soon, however, as a widening of the outlets to the sea diminished these favourable circumstances, the vegetation came to an end; the violent movement of the water near the outlets destroyed part of the new foundation, and the period of sand and clay deposits returned. In this way had the two processes succeeded each other: at the beginning of the present era the second stage of peat-formation had come to an end, and the outlets to the sea were again widened. The alluvial soil of the Netherlands had been formed layer upon layer, and the present state of things could be explained only by assuming a sinking of the soil. This sinking had also been noticed in historical times, and had been found to be

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at the rate of about 18 centimetres (6·45 inches) in a century. Of course, the formation of the alluvium had not taken place in perfectly regular horizontal strata: disturbances had occurred. The diluvial surface had not been flat; and even now old diluvial hills pierced the alluvium in various places. In places which lay deepest there had been no opportunity for vegetation during the formation of peat, and only a sediment of slime had been deposited. Such a hollow in the alluvium was the Zuider Zee, which had been formed thus, and not by floods in historical times, as had been formerly thought, and as the Author stated. At first the inhabitants probably began to live on alluvial soil only during the summer, drawing back to the higher diluvium in winter. Dams were made around the camps in the lower regions as a protection against summer floods, and the soil within the camps was raised by rubbish and by the ooze left by winter floods. Gradually these parts—which were called *terpen*, or *wierden*, as mentioned by the Author—became so high that at last they offered a safe residence, even during inundations. As the population advanced in civilization, and began to apply itself to agriculture, it became desirable to protect the highest parts from the floods, and in that way the system of dams originated. The drainage of the low-lying parts could not begin until the development of the population had still further advanced—not until the windmill was invented,¹ whereby the inhabitants were no longer limited to natural drainage. This was a rough sketch of the origin of the Dutch polder-land, in regard to which, as was evident from its historical development, very complicated conditions existed, especially in the matter of drainage. The meaning of the word “polder” did not seem to be very correctly explained in the Paper. A polder was a piece of land which could be drained naturally or artificially, it being protected from the water outside by a dam, or partly by a dam and partly by high ground. For administrative purposes the term was sometimes applied to a combination of different polders. Areas of alluvion outside sea-walls were termed *kweelders* in Groningen, and *gorzen* or *schorren* in South Holland, Zeeland and North Brabant; when their level had become sufficiently high, they were *ingepolderd*, that was, dammed in or reclaimed. When, however, ground always covered by water was reclaimed, the polders thus obtained were called *droogmakerijen*. Legends about disastrous floods should always be received critically, as much had been added to them in the course of centuries, e.g., the inundation of the Dollart had not caused any loss of land along the Frisian coast, and

¹ By Floris van Alkemade (slain at Stavoren).—B. van K.

as already stated, the Zuider Zee had not been greatly enlarged in historical times. The description on p. 5 was not very clear in regard to the discharge of water from the polders. If a polder lay high enough to discharge its water at low water, the discharge took place in one or more places chosen with reference to the direction of the wind and the water-channels outside. To facilitate the outflow of the water, wide canals and ponds in front of the sluices, if the water came from different sides, were useful. If the polder lay too low to discharge its water naturally, pumps, formerly driven by wind, but now by steam, electricity, or gas, were employed. Further, in front of the pump a reservoir was required for the water when it came from different directions; definite changes in the dimensions of these reservoirs in passing from the north to the south of the country did not occur. To the section on "Polder-Boards" should be added that the Provincial Government had supervision of those Boards and the Queen the supreme supervision; also that the Crown appointed the members of the Boards of those *waterschappen* or polders (hydraulic societies) which supervised protection against the sea and flooding of the large rivers.

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The standard level in Holland known as "A.P." practically corresponded with the ordinary high-water level at Amsterdam before the Y was divided from the sea by a dam and sluices. The Y, on which Amsterdam was situated, was a branch of the Zuider Zee. The level of the different inland waters depended on their situation and their use; and as there was a great difference between the levels in the different parts of the country, there was no question of a fixed general water-level. He did not quite understand what was meant by "compensating reservoirs" (p. 6); and further, Mr. Pennink's investigations did not bear on polder-water. Perhaps the words "to divert part of the outflow from the irrigation-canals . . . into the dunes" referred to the idea of drawing water from the Rhine, pumping that water to the dunes, and using it there to eke out the water withdrawn from the dunes. That idea, however, had been abandoned, as the Amsterdam Town Council did not yet believe the water-capacity of the dunes to be exhausted. Water flowed from the dunes to the polders, so that, when dune-water was pumped away as drinking-water, the polders were relieved of this. The brackish water in the dunes was not found everywhere at the same level, but its depth certainly exceeded 33 to 66 feet.

The whole system of maintaining the dunes, which was peculiar and, as the Author said, an important branch of the art of the Waterstaat engineers, was worthy of being considered more in detail. In speaking of the maintenance of the dunes a distinction

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should be drawn between the coastal range of dunes, namely, those which formed a direct protection from the sea, and the inland range. Seen with the eyes of an engineer, the inland range was of little use except as a freshwater reservoir. But if these dunes were not protected, vegetation did not take root, and the masses of sand drifted according to the direction of the prevailing winds (which in Holland was landward) and covered valuable ground. This was due not to the regular covering of the fields with sand, but to the fact that the tops of the dunes were blown off, and the sand fell down on their lee side, thus causing a gradual drawing back of huge masses of sand. The German word *Wanderdünen* well expressed this condition. Against such a shifting of dunes, artificial works (even trees, which the Author mentioned) were powerless; it was a case of "*gouverner, c'est prévenir.*" The inland dunes must be fixed, in order to protect the interests of the fields lying behind them.

For binding the shifting sand, helm-grass was very effective at first, because it grew well under the existing conditions. When once the dunes were fixed, however, helm-grass—especially if newly planted—often died. Different circumstances might favour the growth of the grass, e.g., new-blown sand, which could be admitted to a certain degree, moisture, and heat; also the removal of things which might choke the plant (such as its own withered leaves) and of superficial vegetation, the roots of which did not go deep enough to bind the sand, etc. Such removal of hurtful elements might, for example, be caused by fire: then the helm-grass would germinate again. In a dune, once fixed, there would always be spots that could scarcely be kept green, especially where the soil was poor, as on the North Sea islands. There it was necessary to have recourse to thatching; and, as this had often to be renewed, it became a rather expensive way of binding the sand together. With the advance of forestry, the idea of wooding the dunes had arisen, and in Mr. van Kuffeler's opinion useful results might be achieved in that way. It was possible that planting woods would not be profitable everywhere, but the saving in the expense of maintenance of the dunes should be taken into consideration, and also the advantages to the climate from the planting of trees. Valleys in the dunes became much more fertile in that way, and he had seen such valleys in the island of Texel transformed into meadows which, he was told, yielded 10 per cent. on the capital sunk in them within 2 years after their first cultivation. He himself had planted dune-woods, beginning at Vlieland; and it would pay there to plant alder trees, taking into account the high prices of fascines in that island.

These questions, however, belonged more to the domain of the forester than to that of the engineer, so that it was quite right that the exploitation of dune-woods in Texel, Vlieland and Terschelling was now in the hands of the State Forestry Department. On the other hand, the maintenance of the coastal range of dunes—the direct protection against the sea—was exclusively the work of the engineer. The conditions not being permanent, one went forward and backward, according to what the sea gave or took; no expensive plantations could be made, but helm-grass thrived in the ever-shifting sand. It was the best material there, because it had deep roots, and because, even while arresting a great deal of sand, it grew so quickly that it seldom became permanently buried. Helm did not grow sideways, and so it did not mat the surface as stated in the Paper. The resistance met by the currents of air in the helm-plantations diminished the speed of the air so much that the sandy surface was not affected by it. The deep roots were also of great value when a storm had wrenched away part of a dune, because they prevented the further dispersal of sand from the slopes, and because dunes which were too high could be made to disperse without any danger of the engineer not remaining master of the situation, as the roots grew again without help. An important principle in the maintenance of dunes was that Nature must never be fought against, but must be led. The artificial moving about of sand was child's play compared with Nature's work on the millions of square yards of sand on these coasts, and the only thing that could be done was to make her bring that sand to the places where it was most wanted, and in such a form that in case of storm and floods the sea would take away as little as possible of it. The struggle was a continual alternation of gain and loss: when the weather was calm, the sea gave sand; when there was a storm, it took it. The engineer had only to take care to fix and keep upon the beach the sand blown by the wind. To fix the sand more effectually, straw or rushes and thatching, or else fascines, were used; whereas helm-grass was best fitted to arrest the sand. In his opinion the endeavour should be to make the foot of the dunes as flat as possible, thus rendering it difficult for the sea to wash away sand during a storm; and should sand be washed away, it would at all events have been useful in exhausting the strength of the waves. If the foot of the dunes had once been washed away, so that the slope rose sheer from the beach, it was customary in Holland to level those dunes to a slope of 1 in 2 and then to plant them all over. In Figs. 2¹ and 9, Plate 1,

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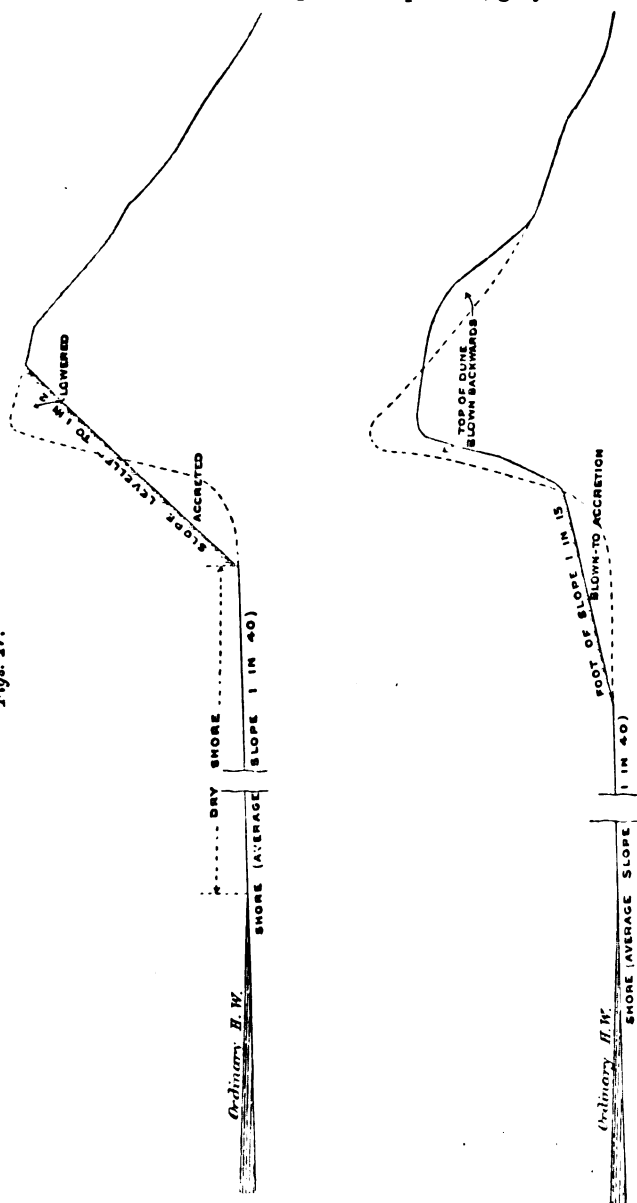
¹ In Fig. 2 "high water" should read "highest water."—B. van K.

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were given instances of sea-dune profiles which had to be protected by slopes, but the figures gave an erroneous idea of the Dutch sea-dunes in general. Such artificial protection was very expensive and, save in exceptional cases, superfluous. The foot of the dune could never be preserved unless it lay above the usual high-water level; otherwise it would diminish at every high tide and could never increase, through lack of a stretch of dry sand in front of it. On the top and on the levelled slope some sand was sure to be gained during the year. Personally, however, he objected to the levelling of slopes, because it was expensive and did not leave the sand in a condition to exhaust the power of the sea on it afterwards. Besides, it seemed to him absurd to bring sand to the sea; in its angry moods it was sure to take enough. He himself, therefore, never allowed the sea-dune slopes to be levelled, except in a few experimental cases. Holes made by the wind dispersing the sand did not cause trouble, because the roots of the helm-grass protected the dunes sufficiently. On high steep crests the grass was cut off, and the sand was blown either landward to strengthen the range of dunes at the back, or else seaward to form the reserve that prevented a breach in a bad storm. In the long run a steep seaward slope was either levelled backwards or it became more sloping. The artificial improvements, therefore, should be restricted to raising the foot of the dune, for which two or three coverings of rushes or straw a year served. Every following covering lay on the preceding one, and was extended a little farther seaward. The part of the foot which might be preserved during a storm could be permanently bound by means of helm-grass. During the winter the foot of the dune lost material, but what was lost then was won back during the rest of the year. The two systems were illustrated diagrammatically in *Figs. 17*. It should always be taken into account, however, that the engineer could never cry "Halt!" to the sea by means of dunes, but must advance or retreat with the coast-line; and only temporarily during a storm could sand keep the sea too busy to penetrate inland. In order to effect this, there should always be present along the coast such a mass of sand that the power of the sea became exhausted upon it. For preventing permanent loss to the coast, recourse must be had to other measures, such as coast-defences, unless the dunes were turned into dams as at Westkapelle and at Petten. The coast-defences there consisted of comparatively long groyne, laid at right angles to the shore, and extending from the foot of the dunes to about low-water mark. The function of those groyne was partly to exhaust the power of the sea, and partly to prevent the sand on the beach from shifting. Groyne did not influence the currents of

the sea; breakwaters did. Up to the present, groynes had been Mr. de Blooq
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Fig. 17.



made exclusively of fascines and stone; on the "de Muralt system,"

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only a few less happy applications had been made, as illustrated in Fig. 10, Plate 2. This system had been abandoned at the Hondsbosse dam.

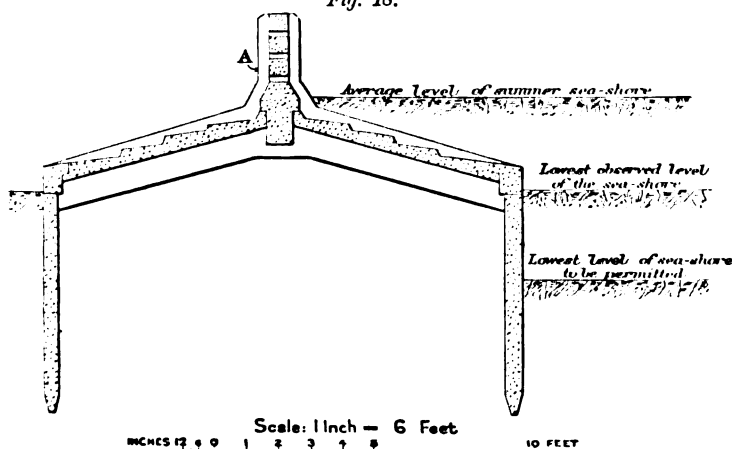
He wished to refute the implication that all applications of reinforced concrete for coast-defences should be termed de Muralt systems, as this was entirely wrong. Mr. de Muralt had done highly meritorious work; he had executed in a very practical manner his ingenious applications of reinforced concrete in defence-works; and he had been the first in Holland to succeed in getting such work carried out on any large scale. This had been a good thing for Dutch engineers; but Mr. de Muralt's "system" was restricted to "beam-slopes," "pin-slopes," reinforced-concrete mattresses and the raising of the breakwater named "Het Ossehoofd," though he had also made, for raising dams, reinforced-concrete walls which were not of any special system. It was not correct, therefore, to call Mr. Sanders's plan for the embankment in the Zuider Zee an application of the de Muralt system, as the Author did. The idea of using cylinders and shields was Mr. Sanders's, certainly not Mr. de Muralt's. The Author also mentioned the application of the de Muralt system for groynes; except in regard to Fig. 10, Plate 2, this was also erroneous. Mr. van Kuffeler and his colleague Mr. de Groot, at Alkmaar, were engaged in making those parts of groynes that lay nearest the dunes of reinforced concrete, though only as an experiment so far.¹ His own project was based on the following considerations. A groyne was really an artificially fixed part of the beach; if reinforced concrete were used—a material which, unlike stone, could hardly be altered afterwards—care should be taken that, in case of unexpected lowering of the beach, the groyne was protected from falls. In the first place, therefore, he made two reinforced-concrete sand-proof walls to form the sides of the groyne; these walls, of course, should be united at the top by beams, which should be connected together at intervals. The space between the walls, which was divided into squares, had to be covered; this might be done with one slab, but as the groyne was highest in the middle, and the slab must have the form of a saddle, in the event of the soil underneath giving way, it would be unsupported. The slab was therefore broken in the middle, and the two halves were kept together with a frame such as Mr. de Muralt used in his beam-slopes. Steps had been formed in the slabs as an experiment, the advantage of them being that the water streaming off was continually broken,

¹ *Post*, p. 54.

and would therefore have less effect on the beach ; but the danger of damage being caused by ice or wreckage was greater. The position of the surface of a groyne depended on the height of the beach, and this varied considerably: in Vlieland the maximum was 1·5 metre (4·9 feet) in the most unfavourable places. Hence it became desirable to vary also the height of the groyne. At the joints, therefore, reinforced-concrete piles (A, *Fig. 18*), with grooves were placed ; and, between these, boards could be brought up to a height of 0·6 metre (2 feet). It was true that in a single point an idea of Mr. de Muralt's had been used, but Mr. van Kuffeler did not consider the work could truly be called an application of that system. He might draw attention also to a defence which

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Fig. 18.



formerly was frequently applied in places that were not too much exposed, like the Frisian coast, namely, screens consisting of poles driven close together and joined by a beam, on the seaward slopes of embankments. They served to break the movement of the waves ; but while this form of defence was effective in many cases, its necessary repairs were expensive. In the island of Urk, therefore, which in its low parts was protected by pole-screens, he had replaced them with a reinforced-concrete structure consisting of piles supported behind and grooved at the sides, between which were slabs. The whole structure rested on the lower parts of the poles of the wooden screen, which had been sawn off below low-tide level.

The following detached observations might be of interest. Groynes were not constructed in front of all dams, but were only found

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exceptionally, in cases of very exposed walls. Basalt was obtained chiefly from the Linz district, Andernach being more the centre for tuff and for trap-tuff. The expense of a basalt slope depended entirely on local circumstances, and the Author's statement that concrete mattresses were much in use was, in Mr. van Kuffeler's opinion, rather too strong. It was remarkable that, Schouwen excepted, none of the Zeeland polder-administrations had used them yet, neither had they been applied anywhere else, though there was a possibility of this, as the Minister had said, when defending the latest budget, that they might be used experimentally for sea-defences in North Holland next year. Fascine-mattresses did not decay, and therefore need not be renewed. They formed a continuous cover, which filled up irregularities of the soil and even accommodated itself to later subsidences where these occurred. Thanks to the great experience of the foremen, a mattress usually sank exactly in the right spot. The loss of a mattress through the turning of the edges was of too rare occurrence to count as a serious objection. In constructing the foundation of the extension of the south breakwater at the Hook of Holland not a single mattress had been damaged. The concrete mattresses of the de Muralt system left at every metre some strips uncovered, though these became dangerous only under very violent attacks. In the event of the soil subsiding, the slabs, owing to their iron connections, no longer formed one mass, but sank pell-mell. Irregularities, such as stones, etc., were a source of additional danger of the slabs breaking or no longer quite covering the soil, which might then be attacked. They were, however, easier to sink, could be sunk in rather stormy weather, and were cheaper than fascine-mattresses. It depended, therefore, entirely on circumstances whether fascine or concrete mattresses were used, but for small works the plant for making concrete mattresses was much too expensive.

The history of the old plans for the reclamation of the Zuider Zee was not important enough to be entered into further. The Lely project, however, might be more fully described, as the Author's description did not go beyond the dam. The dam would shut off the south part of the Zuider Zee from the North Sea; hence thirty discharge-sluices, each 10 metres (33 feet) wide, and two navigation-sluices had been allowed for in the isle of Wieringen. In executing a work of such magnitude, many interests would have to be taken into consideration, as for instance the fishing-industry; but the advantages of the work would be very great. Thus, better drainage of many areas could be obtained, and certain polders could be irrigated with fresh water, as the water in the enclosed

lake would become fresh by addition from the surrounding districts. The work of turning parts of the Zuider Zee into polders would be cheapened, and finally shorter railway communication would be obtained with Friesland and the country beyond, along the enclosing dam. The cost of the dam had been estimated at 23 million guilders (about £2,000,000), but it would amount to 59 million guilders (nearly £5,000,000) if all the works directly or indirectly connected with it were included. In planning the polders, attention had been directed principally towards preserving the canals wanted for navigation, by running the dams in certain directions as regularly as possible, and towards reclaiming, if possible, only fertile soil, without constructing the dams in too deep parts of the Zuider Zee. The average level of the polders had been fixed at about 2 metres ($6\frac{1}{2}$ feet) below the lowest point of the actual bottom of the Zee. Of course powerful pumps had been projected to drain the polders, and the necessary roads and canals for the reclaimed area had been sketched. Altogether, about 200,000 hectares (454,000 acres) would be reclaimed, the estimated cost, after the damming of the Zuider Zee, being 130 million guilders (about £11,000,000). Hence the whole work, which could be completed in 33 years, would cost 189 million guilders (about £16,000,000), without taking interest into account. For a small country like Holland such an expenditure would be a large undertaking, and many considered the enterprise to be not without risk; it was therefore natural that an attempt to restrict it within narrower bounds should have been made. In 1901 Mr. Lely (then a Cabinet Minister) brought forward a Bill proposing to begin by making the dam and reclaiming the two western polders. This project, however, had been dropped after Mr. Lely's resignation. In 1907, when Mr. Jacob Kraus, Assoc. M. Inst. C.E., had taken Mr. Lely's place in the Cabinet, a fresh attempt was made to deal with the matter by proposing a not too expensive beginning, which, in itself, could be defended from an economic point of view, and would inspire confidence in the favourable prophecies as to the results of the whole enterprise. Moreover, it was felt that the execution of the whole work would be furthered by showing that many of the objections raised were without foundation. The proposal in question was to make the part of the dam necessary for the reclamation of the north-western polder, and to reclaim that polder. This meant that provisionally the work would be reduced to one-tenth of the whole enterprise. Should this plan be executed, a reliable basis would be obtained for calculating the cost of the whole work and judging the quality of the reclaimed soil. It was to be hoped that

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Mr. de Blooq
van Kuffeler. then the advisability of executing the whole vast project would be so clearly demonstrated that the Dutch nation would make up its mind to undertake the work which, unfortunately, it was not yet ready to do, after 15 years of only such surety as plans and calculations could give.

Prof. Butler. Professor W. R. BUTLER contributed the following observations based upon experience gained in connection with land-winning and reclamation as carried out upon tidal basins and estuaries on the Canadian coast of the Bay of Fundy, where the conditions assumed in some respects a character of their own. The large areas of alluvium along the margins of some of the rivers discharging into sheltered bays or estuaries of this coast had been recognized by the early French and Flemish settlers of the seventeenth century as open to enclosure and reclamation, and probably experience gained in regions bordering upon those referred to in the Paper had been brought to bear upon its accomplishment. A large portion of the littoral was of Triassic or more recent geological formation, intersected in some localities by eruptive dikes of volcanic rock. The easily-produced detritus from the soft marls and rich soils of this formation—enriched frequently by the decomposition of the trap rock, and also by the sediment of rivers which had drained for ages the adjacent virgin forests—gave rise to accumulations of alluvium of extreme and almost inexhaustible fertility. Much of the early French settlement occurred upon the uplands skirting these rivers and estuaries, and the value of an Acadian holding turned much upon its proximity to marshland, and upon the area of the latter which might be attached to it. Tidal ranges of as much as 56 feet occurred in some parts of the Bay of Fundy, involving a rise of 28 feet above mean sea-level; the erosive power of currents produced by such a wave might be imagined. Under this extreme rise of the tide the rivers and streams were subject to its influence very often for long distances from their mouths; and the impetuous bore or eagre was of regular occurrence in many of them. The vertical accretion on the foreshore seldom continued above a level about 5 feet below that of the highest tides; between this level and that of low neap-tides the surface became covered with a growth of wild grasses, the lower surfaces producing the coarser varieties, with sedges and marine plants. The roots of these formed a sod of considerable toughness. In the absence of regulation or training of either flood- or ebb-channels, erosion and re-deposition were in perpetual interchange: the rushing tidal waters became heavily charged with silt, which was conveyed and deposited in regions of less agitation; the finer and richer particles

were left in the higher reaches of the tide and quiet indentations. Prof. Butler. This deposition could be encouraged in the usual way by groynes and warping expedients, and the enclosure of an area at the head of an indentation might lead to rapid accretion along its own foreshores. Many long narrow estuaries which were probably easily navigable in the days of the early settlement of the country were now such level tracts, bearing grass and cereals. Often the tide was excluded from even its ancient arterial channels, through successive lines of embankment having been pushed seaward by degrees. The course usually followed by the original "dikers" (the word "dike," as in Holland, designating the embankment) was to carry a low dike around any area ripe for enclosure, its course following approximately a level contour, and terminating at the adjacent upland or connecting with similar embankments previously constructed. This "running dike," as it was called, would in general be about 6 to 8 feet in height, with slopes of 1 to 1 when consolidated, and a top breadth of 2 to 3 feet, varying with the degree of exposure. The slopes were quite stable at this angle, if formed of hand-laid "spits" of the alluvium, with the sod outwards, the growth of wild grasses speedily assuring weathering quality. The material was generally obtained from parallel excavations (between which and the dike itself it was important to leave a wide berm); and with the dense, tough alluvium of the higher levels there was usually no difficulty in making a watertight embankment. In exposed situations the seaward face of the running dike sometimes needed protection, and various expedients were resorted to, such as timber piling, stone, or brushwood mattresses. The embankments proper, with labour at 5s. per diem, could be constructed at a cost of about 8d. per cubic yard. In dealing with areas of any extent it was generally found that the direct course of the low embankment became interrupted by creeks or watercourses giving drainage to the land above; these depressions were cut into the alluvium, sometimes almost down to the level of low water. To turn the flank of one of these might involve a detour stretching back to the upland, and it would also generally exclude areas of particularly rich land upon the slopes of the depression. The heavy embankments required to span such watercourses still retained their old French name *abutteau*.¹ Serious difficulty was often experienced with the founda-

¹ The word is found in manuscript spelled in a variety of different ways—e.g. *aboiteaux*, *abateaux*, etc.—and a delightful impartiality is exhibited in its pronunciation. The spelling adopted assumes its derivation from what is said to be an obsolete Norman word, *abuttir*, meaning "to lean against."—W. R. B.

Prof. Butler. tions of these embankments, particularly if the crossing was made near the escarpment of the higher level flats, and treacherous quicksands at great depths were not uncommon. Some settlement was inevitable, and the control of this in the neighbourhood of the sluice, which passed through the embankment, was seldom an easy matter. The sluices were usually large timber flumes consisting of two or more compartments; they were built complete and were then launched and floated into position. Such sluices should be framed as a girder, with a view to rigidity and to the resisting of distortions which would cause leakage or interference with the action of the tidal valves. The valves were usually simple flap-valves of timber, mounted upon gun-metal trunnions. The heavy embankments themselves were built with steep slopes in order to save material and, in the case of the seaward face, to prevent lodgment of the immense masses of ice and frozen mud which floated upon the tide in winter. Brushwood mattress-work was incorporated in the material of the slope, and the freezing of such masses to it exposed the face to serious damage when they were lifted or moved by tide or waves. The seaward face was often given a slope of 1 horizontal to 2 vertical, and the inner face was seldom flatter than 1 to 1. The body of the embankment was bonded together laterally by mattress-work of green brushwood (spruce, fir, birch, or alder—a proportion of the latter two giving it great tenacity). The brushwood was laid by hand at right angles to the line of embankment, and was roughly interlaced to a depth of 7 to 9 inches; it was then compressed and secured by long parallel binding-poles, spaced at regular intervals and breaking joint longitudinally; these poles were cross-pegged by stakes driven well into the underlying course of alluvium. The mattresses extended from the inner and outer faces, alternately, for a distance of about two-thirds of the width of the embankment; longitudinally they were carried from end to end. Between two vertically consecutive mattresses was laid a course of alluvium 2 to 3 feet in thickness. In its earlier stages the work could be done only in short shifts, and, as height was gained, a severe scouring action had to be met as the tide passed over the new work: operations had therefore to be so timed as to ensure this being thatched by the mattress construction. Where this had been accomplished successfully the loss of material was generally trifling; it was often very surprising what a comparatively insignificant covering was sufficient to deaden the scouring effect of a powerful current. Mattress-work with a certain amount of stone filling was employed very successfully in the construction of aprons at the mouths of the sluices, the discharge from which commonly

took place under a considerable head. The alluvium used in "abut- Prof. Butler.
teau" construction was of the character already described; it was usually excavated with the spade and wheeled to the place of deposition, where it needed to be carefully spread in layers about a single "spit" in depth. The cost per cubic yard of finished embankment of this class of work would usually be five or six times as great as the cost of the running dike. These heavier embankments were more exposed to injurious wave-action, and as their upkeep frequently became expensive, timber and stone cribwork was sometimes used for a facing. The timber was practically indestructible—at all events, up to the level of neap-tides—and in these silt-laden waters it was attacked neither by the teredo nor by the limnoria. He was not aware that any attempt had yet been made in that region to build an embankment of homogeneous material and with sufficiently gentle slopes to enable it to resist (with moderate protection) the wave-action to which it would be liable in exposed positions subject to the rise and fall of tide experienced there. It was probable that in the provinces of Nova Scotia and New Brunswick upwards of 120,000 acres of marshlands had been reclaimed, and were defended by works constructed by these methods.

Mr. W. DYCE CAY stated that he had had several good oppor- Mr. Dyce Cay.
tunities of studying the protection-works of Holland, and in 1894 he visited the seashore from 's Gravenzande to Ter Heiden near Monster, making sketches and measurements of the embankment. The crest of the sand embankment was about 30 feet above high water; on the seaward side the slope for 45 feet measured horizontally was covered with bent-grass; the next 40 feet towards the sea, extending to high-water mark, had reeds or rushes planted in small bundles 2 feet apart each way: these two distances in relation to the height made the seaward slope nearly 3 to 1. In the landward direction the top of the bank measured 65 to 70 feet from the crest, with a fall of 5 feet from it; thence there was the usual earthwork slope with a fall of 15 feet to the flat ground. At some little distance back there was another bank, parallel to the main one, thus doubling the protection. In the sea, extending from the foot of the bank, there were groynes about 300 yards apart (measuring along the coast) and 273 yards long: these appeared to be founded on mattresses, as wattles were visible projecting out from the base. Above and through them the whole area of a groyne was covered with small piles, placed in longitudinal and transverse rows. They were of unsawn fir 6 to 10 inches in diameter, the larger ones being at the outer end of the work,

Mr. Dyce Cay. and placed 12 inches apart from centre to centre in the rows. There were six rows longitudinally in the outer part, and seven rows of the smaller piles in the inner portion, above low water. The transverse rows were 7 to 10 yards apart. Between the piles the structure was filled with rubble, with rubble pitching on the top surface, the stones of the pitching at the outer part being squared and 2 to 3 square feet in surface area. The cross section of the groyne at its middle was hog-backed, 42 to 45 feet wide, and with a camber of 6 feet at the centre. An interesting feature was that at one part of the coast the groynes had a row of pile-heads standing $5\frac{1}{2}$ feet above the stonework along their centre ridge; these were root ends of trees, 10 inches in diameter, and spaced about 17 inches apart. The object apparently was to collect the sand, driven by the wind, near the root of the groyne, which acted like the snow-fences on the Highland Railway in Scotland. On his own works he had generally placed a line of piling in the bank parallel to the shore-line, the head of the piles being flush with the slope of the pitching, and as little above low-water level as convenient. This provided a foundation or abutment for the upper part of the slope. In 1868-70 he used this plan with economy and success on the left bank of the diversion of the River Dee at Aberdeen,¹ where the piles were of round Scotch fir, not less than 8 inches in diameter at the small end, 12 to 16 feet in length, and placed 18 inches apart. The revetment was of granite rubble pitching, laid on small stones and gravel, no stone of less weight than 56 lbs. being used. At the "cut off" or crossing of the old bed of the river, he introduced a rubble deposit in front and a clay wall behind the piles. On the esplanade works at Lerwick he had been obliged to protect the part of the slope below the piling with large concrete bags, on account of the run of the sea in storms. With regard to the sand-dunes protecting the coast, he considered that the sand came from the Rhine and the other great rivers discharging at the Dutch shores; the sea deposited it on the beach, whence the wind carried it inland and formed the dunes, as on the English coasts. There was thus an endless supply of sand. In conclusion, he would refer to an interesting lecture on the progressive development of Holland delivered by the late Mr. J. F. W. Conrad, Inspector-General of the Waterstaat, as President of the International Navigation Congress held in 1894 at The Hague.²

¹ Minutes of Proceedings Inst. C.E., vol. cxxxi, p. 79.

² Report of the Proceedings of the Sixth International Navigation Congress, The Hague, 1894, p. 18.

Mr. H. CLEGG remarked that the de Muralt system, which claimed Mr. Clegg. searching investigation, was, to his mind, the outcome of much careful thought and ingenuity, and it marked an epoch in coast-defence work. He understood that the inventor had really three systems of protection-work: the first was a method of protecting the sea-bed adjacent to the coast-line, and the other two were methods of protecting the slopes or foreshores of the dikes. Mr. Clegg looked in vain for any enlightenment or recommendation as to the utility of the two latter methods in England. Would the Author suggest that concrete mattresses on the de Muralt system were applicable only to such a coast-line as that of Holland, where "falls" of the sea-bed were numerous; or did he consider that they might be applied to some parts of the east coast of England where there were no falls, but an ever-moving unlimited quantity of shingle and sand below low-water level? Again, would the Author advise the adoption of the two protective systems for foreshores above low-water mark? Mr. Clegg's own impressions were decidedly in favour of the de Muralt systems; the system of concrete mattresses, and the heavier type of foreshore work constructed simultaneously, seemed to be almost ideal for coastal protection, but unfortunately they were beyond the reach of local authorities dependent entirely on local rates. He had good hopes that concrete work for foreshore-protection would be generally adopted in the district under his supervision at Felixstowe and Walton, seeing that quite recently he had brought before the Committee portions of two piles which had been practically eaten away by the destructive teredo, although only driven in the foreshore about 18 months before.

Mr. G. E. W. CRUTTWELL remarked that the pitching of the Mr. Cruttwell. typical dams protecting the coast of Holland was described by the Author as being laid upon a layer of clay, and the pitching of the Essex sea-walls under his supervision was similarly described. No doubt clay formed a suitable substratum where the water-level next the pitching was not subject to much variation; but with a rapidly falling tide there was a danger of the water impounded at the back of the clay exerting an outward pressure, and thus producing a movement of the pitching. It was questionable, therefore, whether in some instances a more pervious material, such as gravel, would not make a better foundation for the pitching than clay, and he would like to have the Author's views on the matter.

Mr. H. VAN GELDEREN, of Walcheren, noticed that on p. 14 considerable lengths of sea-frontage were stated to have been carried out on the de Muralt system on the island of Walcheren, and he wished to point out that protection on that system in Walcheren

Mr. H. van Gelderen.

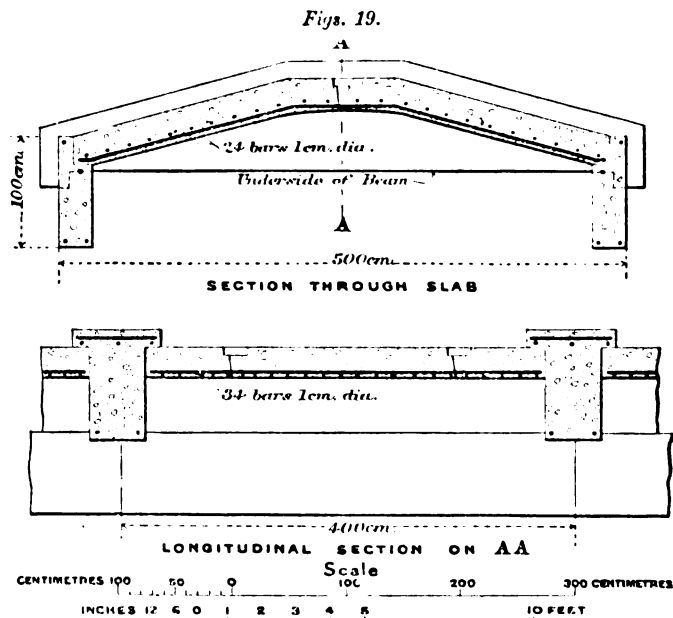
Mr. H. van Gelderen. existed only at Domburg, where the foot of the dunes above high-water level was covered for a length of 130 yards. At Flushing, on the River Scheldt, about 110 yards was thus protected.

Mr. A. T. de Groot. Mr. A. T. DE GROOT, of Alkmaar, stated that, to prevent erosion on the west coast of North Holland between Helder and Petten, the whole sea-shore had to be protected by groynes about 150 metres (492 feet) long, spaced about 200 metres (656 feet) apart. The construction of these groynes was commenced in 1899, and twenty-eight were now ready: thirty-six remained to be built before the whole coast would be defended against the sea. The groynes were laid at right angles to the coast-line, and were built of fascines and basalt from the Rhine. Such works were very expensive, the cost of one groyne being about £3,000, exclusive of the cost of the necessary railway in the sand-dunes along the coast. Recently the question of the use of reinforced concrete had been considered. In general, however, it seemed necessary that concrete structures should be built in dry situations. The engineer was therefore obliged either to adopt a method of construction that could be carried out on the site itself, with or without making it dry, or else to choose one in which separate parts were made elsewhere and placed together afterwards. The principal and most expensive part of groynes was that situated seaward of the low-water line, and as it was not possible to keep this place dry, it was only possible to use there the type of construction with separate parts. For underwater work the system of Mr. de Muralt was not considered suitable. It might, perhaps, in some circumstances, take the place of the excellent Dutch fascine-work, for covering slopes, because it was less expensive; but it was unfit for work of large volume, such as was necessary in building groynes. If separate parts had to be used they must be supplied and placed at as small a cost as possible, and thus it became necessary to make them so small that they could be handled without difficulty, or to give them such a form that they could be floated to the site of the work. But a structure whose separate parts were small would not give good results under water, because concrete had a low specific gravity under water, and also because it was difficult to effect good connection of the parts under such conditions. The construction of floating sections, in the form of caissons or cylinders, had often been applied to quay-walls, but, apart from the heavy cost of using these in the open sea, they were unfit for groynes. The dimensions of the caissons must be fixed at least 4 weeks before the time of placing them in position; and that was not always feasible, as the sandy bottom of the sea was always changing. The foundation of a groyne must be placed on a mattress

of fascine-work to prevent the sand underneath from being washed out and carried away by the waves and currents; and it was also necessary to prevent the heavy and expensive stones from sinking down into the bottom of quicksand. It was thus clear that, so far, fascine-work was to be preferred to concrete for the parts of groynes under water and seaward of the low-water line. Fascine-work, which was really rendered necessary by the existence of a sea-bottom of the nature of quicksand, had the great advantages that it could be changed at the last moment, and that, by reason of its great elasticity, it followed all changes in the sandy bottom; moreover, it was easily repaired after damage by storm. If concrete was to be used with success for groynes, it must be limited to the part of the work landward of low-water line, so that the saving would not be very great. For instance, the most seaward part of a certain groyne (that was, the head and a length of 20 metres) cost £1,000; the middle part (95 metres) cost £1,650; and the landward part (35 metres) cost £350. The cost per lineal metre thus increased considerably the farther out one went. The three groynes which he had constructed during 1910 between Helder and Petten were partly of reinforced concrete. Each groyne had a length of about 150 metres (492 feet), of which 53 metres at the landward end was of concrete. This part was 5 metres (16 feet 5 inches) broad, the difference in height between the top and the edge was 50 centimetres (20 inches), and the longitudinal slope was the same as the slope of the shore, averaging 1 in 45. A system of beams and slabs was used, as shown in *Figs. 19*. Each section of the groyne was 4 metres (13 feet 2 inches) in length, and was formed by first erecting in trenches the longitudinal side beams shown, which were made in the neighbourhood of the work. The sea-shore between the beams was then heaped in a round-topped form, and covered with straw or rushes. Afterwards the concrete slabs, reinforced with bars each 1 centimetre thick, were made between cross screens, the slabs being $3\frac{1}{2}$ metres (11 feet 6 inches) long. After the concrete had set, the space between two slabs was filled with a cross beam, which had a framed flange with a breadth of 80 centimetres (31 inches) and a height of 15 centimetres (6 inches) above the slab. This again was reinforced with five iron bars, of which one was bent round the bar of the beam in the direction of the length of the groyne; and in addition the cross beams bound together the side beams as shown. In this way all the beams were bound together inwardly, and sinkage of a part of the structure could take place without injury. In order to prevent scouring out and subsidence of the structure, the sandy

Mr. A. T. de Groot.

Mr. A. T. de Groot. sea-shore next the groyne was protected with fascine-work and basalt over a breadth of $2\frac{1}{2}$ metres (8 feet 2 inches). The groynes of this type had cost about £3 8s. per lineal yard, and were



satisfactory; they were 3 months old. In 1911 four groynes of this design, somewhat modified, would be built, in continuation of the defence of the west coast of North Holland.

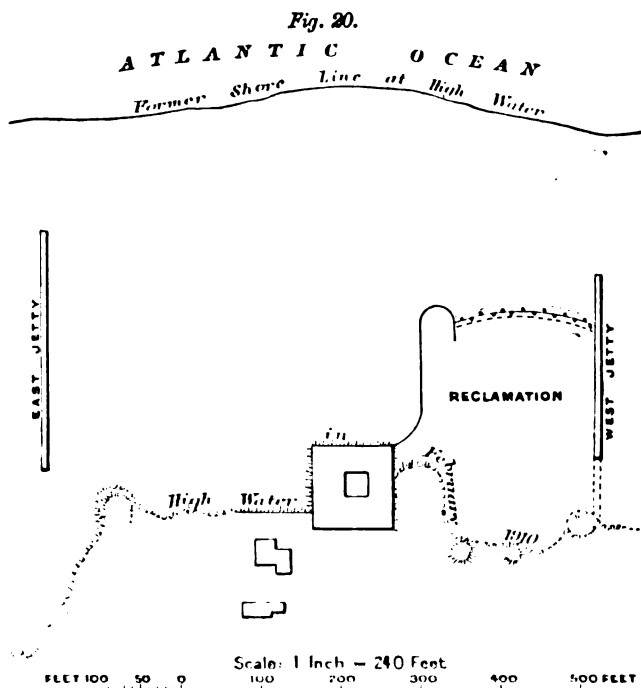
Mr. Haupt. Mr. LEWIS M. HAUPT remarked that after more than six centuries of experience, the Dutch engineers had justly secured a world-renown as maritime engineers; yet in their early works it appeared there were some disasters due to exceptional storms which destroyed 20,000 lives in several instances, but which served to point the way to greater successes. This interesting history, briefly summarized, indicated that at the mouth of the River Ems, near Groningen, a total of about 69,000 acres had been reclaimed in about 350 years, giving an average of about 200 acres per annum. Again, the Author's statement that the reclamations effected at the mouths of the Maas and the Scheldt since the twelfth century had increased the agricultural area of Holland by about 250,000 acres, would give an average yearly increment of 312 acres, or less than $\frac{1}{2}$ square mile per annum, if continuous. The cost of draining the

Haarlem Lake was said to have been £14 per acre, while the cost of Mr. Haupt. maintaining 50,000 acres in the polder of Schouwen was at times as high as £25,000 per annum (p. 6). These data might serve as a guide to the cost and results of the irrepressible conflict between man and Nature under exceptional difficulties, and where the ever-active marine forces were tending to destruction rather than to conservation. In some instances the Author had pointed out the durability of the works as compared with similar works under other conditions of exposure; but that was no criterion, as the conditions were so variable. It was worthy of note that the longitudinal embankments or dikes usually followed the contours of the coast, and were placed between low and high water, rising through various profiles to an elevation supposed to reach so far above wave-action as to protect the backing from the downfall of the arrested wave. These barriers were frequently supplemented by groynes, placed at short intervals normally to the coast-line, in order to protect the footings and impound the littoral drift; but the general consensus of opinion appeared to be adverse to them, as their cost seldom justified the means. Such structures were common both in Europe and in America, especially at sea-side resorts, where they were unsightly and interfered with the continuity of the beach for driving. Good examples of the use of these groynes might be seen at Felixstowe and elsewhere. In view of the heavy cost and slow progress indicated in the reclamation of the marshes of Holland, it might serve a useful purpose to direct attention to the extensive deposits which occasionally took place at the delta-mouths of sedimentary rivers. For some inexplicable reason these deposits appeared not to be fully appreciated, since the effect of natural laws in creating such extensive contributions to the rich arable lands was being arrested by dams confining the silt to the bed of the stream. In a study of the several passes or mouths of the Mississippi River, made by Mr. Haupt recently, it was noticed that the main stream had breached its natural banks by cutting crevasses at "The Jump" in 1841, at "Cubitt's Gap" in 1863, and at "Pass à Loutre Crevasse" in 1891. This portion of the river was estimated to be discharging about 250 million cubic yards of sediment annually to the Gulf; and the United States Coast-Survey charts gave the lines of the shore in 1854 and in 1884, with soundings, thus affording the necessary data for computing the accretions through the first two of the crevasses during the 30 years. From these data it was found that the sediment escaping through the upper breach amounted to 35 million cubic yards per annum, and had added 113 square miles, or 72,320 acres, to the fast land. Its mean depth was 3 yards, and

Mr. Haupt. the cost of creating this land, by hydraulic dredging, at 10 cents per cubic yard, would have been \$105,000,000 (£21,000,000). A survey of Cubitt's Gap made in 1893 showed a total accretion of about 50 square miles of land, representing an annual deposit of 13 million cubic yards, which at the same rate would have cost \$40,000,000 (£8,000,000). The navigable channel was not affected injuriously by these openings in the banks, which relieved the river of 18 per cent. of the sediment that must otherwise have been carried through to the Gulf and deposited on the bars at the mouths of the passes, thus extending them seaward, reducing the slope, and increasing the flood-heights of the valley for many miles. The bars were found to be moving seaward at the rate of 1 mile in 20 years. No recent surveys of the deposits at the Pass à Loutre were available, but the fast land was rapidly extending into Gardner's Bay, and adding to the available territory of the State; yet these several crevasses were being closed, under the impression that it was necessary to improve the depths of water over the outer bars of the South and South-West passes, which were being "improved" by jetties and dredging.¹ In this connection it might be in order to inquire whether it would not have been possible to utilize a large portion of the 130 million tons dredged from the Mersey bar and channel at Liverpool in the reclamation of fast lands near the site of the work. From the foregoing statements it appeared that the reclamation at the delta of the Mississippi, by natural processes, was about twelve times as rapid as in Holland, and that the recovery of the land cost nothing, while the navigation was benefited by the reduction of silting. The Paper, in fact, seemed to be confined to the methods of defending riparian lands from erosion rather than to the utilization of the maritime forces for creating deposits, which was of greater importance, and to which, in Mr. Haupt's opinion, too little attention had been paid. It was true that much had been recorded by physicists and engineers on the subject of the movements of littoral drift, and many devices had been tested for the defence of alluvial coasts from erosion by embankments, spurs, groynes, etc.; yet it would seem that there was room for greater efficiency in the utilization of the resultant of all the forces, operating through long periods. This might be accomplished in such a manner as to cause the dynamic forces of breakers moving at an angle to the contour of the shore,

¹ For a more complete consideration of these special problems, see a Paper on "The Mississippi River Problem," *Proceedings of the American Philosophical Society*, vol. xliii (1904), p. 71; also *Cassier's Magazine*, vol. xxxv (1909), p. 722.

augmented by the action of the flood-tides and currents, to propel Mr. Haupt. the sediment, with which the waves became charged in breaking on sandy beaches, shoreward and deposit it within restraining enclosures. There would thus be reclaimed, in a short time and at little cost, lands which had been seriously eroded by the same seas. It was not possible, in a brief discussion, to describe instances of the application of such methods in various localities, since every case called for distinct treatment; but one of the most recent examples in the United States might be cited as a particularly



desperate case. At a summer resort known as Edgemere, on the south coast of Long Island, nearly the entire property had been cut away, in consequence of long spur jetties, built on either flank of the tract in question, robbing the beach of the littoral drift. The low-water line had receded more than 600 feet, causing a loss of about 8 acres from the three properties lying between the jetties. The spurs had been built in the hope of recovering the ground, or at least arresting the movement, but this result had not followed, as the sea had flanked the inner ends of these works and continued its devastations, requiring the cottages to be bulkheaded. The problem was to

Mr. Haupt. gather sand from the foreshore, to cause the waves to cast it into an impounding basin whence it could not escape, and to utilize the west jetty as an abutment for the work. As only one of the owners was willing to try the experiment, the works were confined to his frontage: they consisted of a hooked jetty (*Fig. 20*), extending beyond the low-water line and rising to above high water, from the outer curve of which a hurdle of suitable height was sprung on a flat segmental arch (in plan) to the jetty previously built, thus enclosing about $1\frac{1}{2}$ acre, at a cost of \$4,400 (£880). The work was commenced in March and completed towards the end of June, and the effect was immediate, for in a few weeks the high-water line advanced about 200 feet, and by the end of September the area was filled almost to the top of the hurdle. Subsequent reports continued to be highly satisfactory. He strongly inclined to the belief that instead of placing the resistance directly on the "firing-line" on the strand, it was better to force it out to the counterscarp beyond low water and break the waves before they reached the toe of the inner works, thus forming a detached insular barrier promoting deposition and advance of the shore-line.

Mr. Kapteyn. Mr. A. KAPTEYN thought it might be interesting to point out that at the present time there was under consideration in the Second Chamber of Parliament the same Bill for the reclamation of the Zuider Zee as had been presented in 1907 by the Minister of Public Works (Waterstaat) at that time, Mr. J. Kraus, Assoc. M. Inst. C.E. This Bill aimed, as a first step, at the reclamation of polder No. 1 (*Fig. 14*, p. 18), situated to the south of the island of Wieringen, and included, of course, the construction of that part of the enclosing dam connecting the coast of the province of North Holland with that island. The cost of the work in question would, it was estimated, amount to 23 million guilders (nearly £2,000,000), about one-eighth of the cost of the entire Zuider Zee project. For these 2 millions sterling, 41,000 acres of land of very good quality would be reclaimed, and that part of the scheme could be finished within 7 years. Moreover, it was expected that the successful execution of this limited project would give confidence to those who had doubts as to the practicability of the entire scheme, and would give them sufficient encouragement to vote for the completion of the enclosing dam to Friesland, in order successfully to reclaim the other three polders surrounding Lake IJssel.

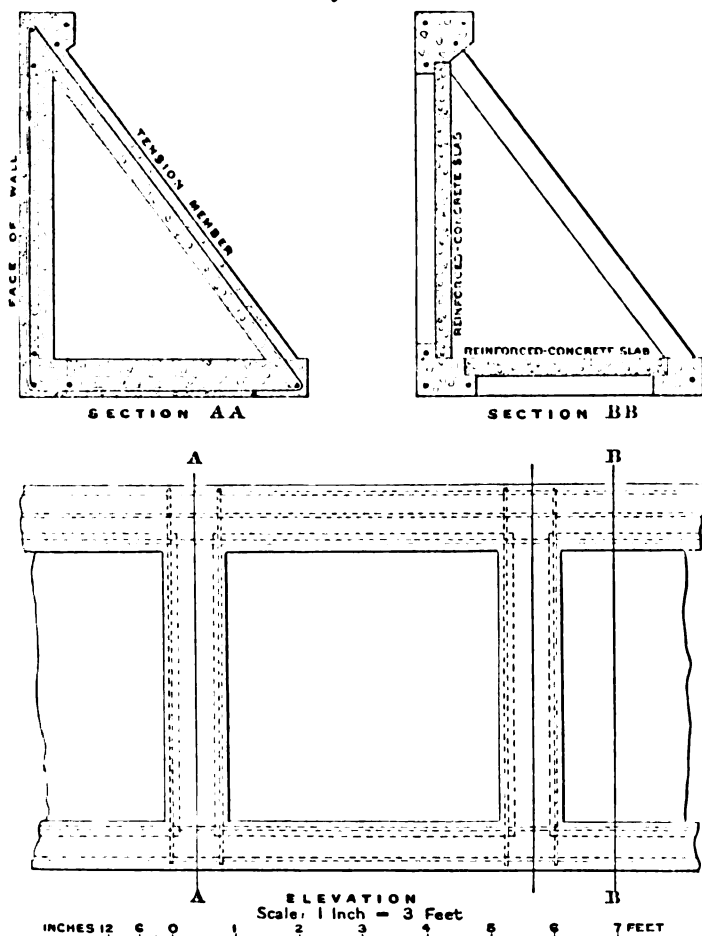
Mr. Latham. Mr. ERNEST LATHAM, who had had the privilege of accompanying the Author on his inspection of the Dutch coast-line, considered that one of the most remarkable sea-defence works there was the Helder dam; and its exposed position, on the North Sea and at the

apex of the province of North Holland, rendered it of great importance to this province, and liable to severe handling by the sea. Taking into account the various historical inundations of the Dutch coast, the portion of the coast-line most likely to be breached in the future was, in Mr. Latham's opinion, the Helder dam. Most of the larger works in Holland were executed at the back of very flat and sandy foreshores, but this dam was one of the few exceptions. The Author mentioned that the underwater slope had a uniform gradient of 1 in 1, and that it ran down at this slope to a depth of over 130 feet. Although several thousand (he believed 4,000 to 5,000) tons of stone from the Rhine had been dumped at random every year from hopper barges at the toe of this work, there had been no appreciable increase in the level of the sea-bed, as the stone was continually slipping forward, or gradually settling into the sand. During the Author's visit to Holland, he was accompanied by Captain N. Stapleton, who was able to criticize the work from a nautical point of view; and, from the conversations which Mr. Latham had subsequently had with him, it appeared that considerable attention was paid by Dutch engineers to the question of navigation, as well as to reclamation, when constructing coast-defence works. In many English rivers reclamation had been carried out largely without regard to the question of navigation. As gradual reclamation proceeded up-country, the volume of tidal influx and efflux grew less and less, and as the scouring currents at the mouth of the rivers were slackened, shoals were formed and bars were deposited, materially interfering with the navigation both there and farther inland. In order to overcome this, training-works at the mouths of the rivers generally became necessary, and they might be observed at the mouths of most Dutch rivers. In addition, therefore, to the expense of reclaiming land which took many years to attain agricultural prosperity, there had to be considered the expenditure necessary at the mouth of the river on whose banks the reclamation was carried out. As the Author explained, the navigation-canals took a considerable portion of the land-drainage pumped from the adjacent marshes; and in the case of the North-Holland canal, the current so set up helped to keep clear the mouth of the naval station at Nieuwediep. This was interesting as showing what could be done by artificial scour. This harbour was much handicapped by having no river-outfall, and the consequence was that the banks and shoals outside the harbour were constantly changing. He thought the strength of the Dutch dams was considerably in excess of those on the east coast of England. Near Flushing he

Mr. Latham. had observed a small schooner of about 100 tons, which had been cast ashore on a basalt-pitched bank; such an occurrence on the shores of Essex would undoubtedly have brought about a collapse at the base of the bank, probably followed by a breach. On several occasions he had had to visit places on the Essex walls where ships had gone ashore, and in some cases the result had been to cause considerable damage to the river-bank. This point, he thought, was one to which sufficient attention had not been paid. In his opinion, the de Muralt systems were better adapted to withstand weathering and the impact of waves than either British or Dutch methods employed hitherto. The Author showed that to face a typical dam in Holland, by the older methods, cost nearly 16s. per square yard. This work, of course, was much heavier than the protection usually carried out on the British coast; and the only expenditure of a like nature in England was on the sea-wall of the Tendring Levels, south of Frinton. This was the nearest approach to Dutch work that he had seen in England, and he thought that its cost probably very nearly equalled the foregoing figure. Usually the cost of rendering a river- or sea-embankment sea- and weather-proof did not exceed 7s. per square yard. He had recently completed the facing of a river-bank at Aldeburgh, on the de Muralt (light) system, and the total cost including royalties had amounted to 7s. 6d. per square yard. The work was of an exceptionally heavy character, consisting of remaking the face of the bank and cutting off piles; it also involved cartage of materials to the site from a considerable distance. In this case it would appear that the cost of thus facing the river-wall was less than it would have been if chalk or ragstone pitching had been employed; and provided the troubles arising from expansion and contraction, which were serious in a long stretch of wall-facing, could be overcome, there seemed little doubt that reinforced concrete would rapidly replace the ragstone and chalk pitching now so largely used in England. In the case of stone pitching, work had to be done on the face of the wall every year, whereas many of the works in reinforced concrete that he had visited in Holland had stood without attention for more than 4 years. Mr. de Muralt seemed to have been the first engineer to realize the value of elasticity in constructional work carried out on open foreshores. The reason why stone pitching had not previously been superseded was doubtless the fact that there had been no available alternative which allowed the facing to settle and accommodate itself to the gradual sinkage and compression of the earth-work behind it. The only other system which had been designed to

meet this requirement involved the use of very small units, whose attachment depended on continuous wiring passing through the slabs. The systems of reinforced-concrete protective works described by the Author were all based on the assumption that a sloping

Figs. 21.



face could be secured to the wall or dam. In England, however, it frequently became necessary to employ a vertical face, and an adaptation of the heavier type of de Muralt construction had already been introduced into this country, the face of the wall being vertical (*Figs. 21*)

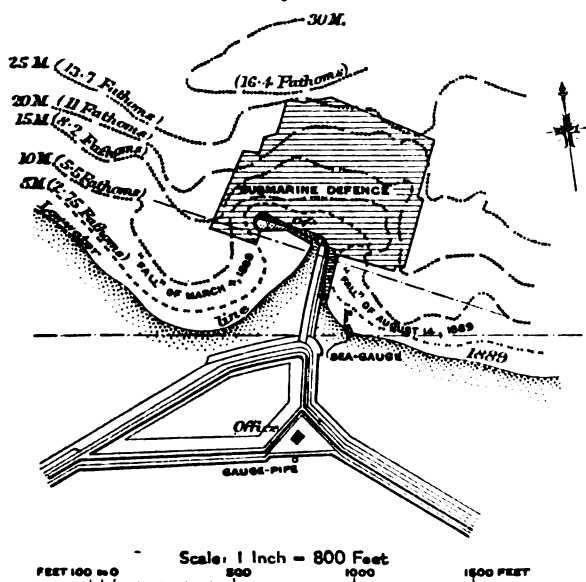
Mr. R. O. van
Manen.

Mr. R. O. VAN MANEN, of The Hague, observed that the "falls" referred to on p. 12, which were exceedingly important for the Netherlands generally and for the province of Zeeland in particular, occurred almost exclusively in this province, in the south-western part of Holland, which comprised, in addition to a continental part, several islands separated by vast estuaries as shown in Fig. 1, Plate 1. In these estuaries there were powerful tidal currents, alternately flowing from and to the ocean, and dependent on the tidal range, which in ordinary circumstances amounted to an average of about 13 feet in the Wester Schelde, which was the branch past Flushing, and of 10 feet 2 inches in the Ooster Schelde, on which Kolijsplaat was situated. These ranges were increased at spring-tides by 2 feet 4 inches in the Wester Schelde, and by 1 foot 4 inches in the Ooster Schelde. When it was considered that the width of these estuaries in some places was more than 3 miles, and that the depth of their beds was 100 feet and more—in some parts as much as 164 feet—it became clear that they might really be considered as parts of the sea. The beds of the Zeeland streams consisted of fine sand which was easily moved by the tidal currents, and a consequent peculiarity of these streams was their changeableness. By consulting detailed maps, such as the excellent charts prepared by the Hydrographic Office of the Dutch Admiralty, it would be seen that the Zeeland estuaries were covered with sandbanks, both large and small, which were liable to change in size and position; and those changes might have injurious effects on the coasts. A sandbank travelling towards the shore would cause a narrowing of the channel between the shore and the sandbank. But in such circumstances Nature tried to win back in depth what had been lost in width: the erosion caused by the tidal currents was intensified, and the submarine slope became attacked. Naturally, the gaining side of the sandbank was not injured by the attack. This unceasing and continually increasing erosion might cause the submarine slope suddenly to collapse or "fall" away. Thus the shores were being always threatened by the sea, and it might be said that an enemy was always standing on guard outside the coastal barriers. Though severe losses of ground had been the result of the erosion just described, yet it was Mr. van Manen's opinion that this did not account for all the falls that had been noticed. There was yet another element to be taken into consideration—an enemy within the barriers. In order to make this clear, attention must be drawn to two things, namely, the nature of the Zeeland soil, and the part played by the hydrostatic pressure of the ground-

water. The upper stratum was alluvial, and, as a rule, the surface of Zeeland was no higher than about the level of high water. Land there was not reclaimed till it was ripe, and such was not considered to be the case until the ordinary high water no longer washed across it, and the surface was only covered occasionally, during spring-tides. Generally such ground lay 12 to 20 inches above ordinary high-water level and was covered with vegetation called *schorgras*. After reclamation, the upper layers of the soil shrank, from loss of water; old polders might be seen lying $6\frac{1}{2}$ feet and more below high water of ordinary tides. Latterly, an extensive

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Fig. 22.

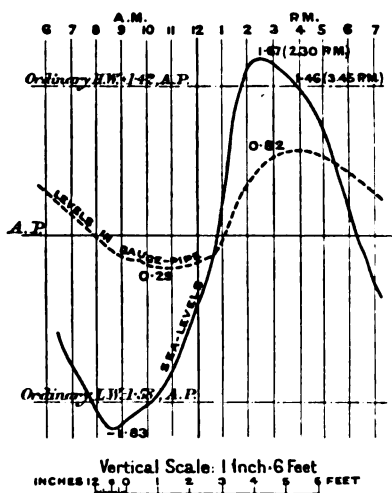


investigation as to the nature of the soil had been conducted, and it appeared that only a few metres at the top had an alluvial origin, consisting generally of clay or clay-like cohesive soil. Under this a not very thick layer of peat and a stratum of diluvium were found—the latter 66 to 82 feet in thickness, and consisting of sand without any cohesiveness and exceedingly permeable to water. Of course the rise and fall of the sea must influence the hydrostatic pressure of the ground-water in the deeper strata of the reclaimed lands, with which—owing to the permeable layers of sand—it was in connection. The less the distance from the sea, the greater this influence would be. Fig. 22 was a plan of part of the Anna-Friso

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polder in the island of Noord Beveland, where a vertical iron tube or pipe for making observations had been sunk in the position indicated. This pipe, which was $2\frac{1}{2}$ inches in diameter, had been sunk to a depth of more than 33 feet, so that it reached into the diluvial stratum of sand. After all soil had been removed from the pipe, the level of the water in it was gauged. Of course this level was not the same as that of the surrounding ground-water in the polder; but it was a measure of the hydrostatic pressure in the layer of sand at the bottom of the pipe, which pressure increased and diminished with the rise and fall of the sea. The observations

Fig. 23.



of the 21st March, 1890, during a full period between flood- and ebb-tide were shown in Fig. 23; these were taken partly every half-hour, and partly every 5 minutes, and gave some idea of the pressures that might occur. The subject was treated more fully in the standard work¹ by Mr. B. G. Hogerwaard, Chief Engineer of the Provincial Waterstaat in Zeeland, formerly engineer of the State Waterstaat. It was this gentleman who had ordered the measurements described above to be taken; and a study of his book and of an essay² of his might be recommended to anyone who wished

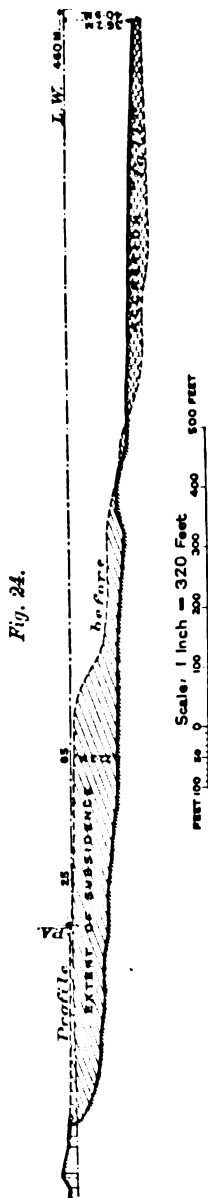
to know more about the subject. It was obvious that during the period of low water a hydrostatic pressure working in the direction of the sea became possible in the deeper strata. As this pressure was considerable, especially when a high flood was followed by a low ebb, the conditions might become dangerous. Cases were known in which the hydrostatic head had amounted to 6 feet, and even 6 feet 3 inches, and the hypothesis that these high pressures played a great part in the causation of "falls" became acceptable when it was considered under what circumstances they took place. These high pressures,

¹ B. G. Hogerwaard, "Memorie over de verdediging van de Zeeuwsche oevers" (Report on the defence of the Zeeland coasts). Middelburg, 1893.

² "De oeververdediging in Zeeland sedert 1860" (Coastal defence in Zeeland since 1860).

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then, were the enemy within the barriers; and perhaps the use of the term "falls" should be restricted to sudden losses of ground due to this cause. Turning to actual examples, *Fig. 24* gave an idea of the important fall in the Vliete polder (in the north of the island of Noord Beveland) on the 11th September, 1889. The fall had a length, measured along the coastal barrier, of about 1,300 feet, the maximum breadth within the low-water mark being about 750 feet. An area of more than 14 acres was thus swallowed by the sea, causing the displacement of 1,233,000 cubic yards of soil. The damage done to the coastal defence extended over a length of 860 feet; and the subsidence penetrated as far as 775 feet westward of the east side of the submarine mattress. Locally the seaward slope of the sea-wall crumbled away as high as and higher than the outer berm; while the eastern slope of the connecting groyne and the end of this groyne slid away up to the summit. The groyne along the east side of the submarine defence disappeared with the foreshore. Mr. van Manen mentioned this fall not only because of its size, but also because it was one of the few described by trustworthy eye-witnesses. The President and the Surveyor of the board of the *calamiteuse* Vliete polder saw at 8 a.m. a fissure, perhaps about a foot in depth, appearing along the water-line; after which more landward fissures, having a length of 30 to 60 feet and running parallel to the water-line, became visible 6 to 12 feet farther back. After the ground had fallen away as far as a fissure, fresh fissures appeared, and the phenomenon repeated itself. In this way the subsidence continued for 4 hours, at the end of which period the dimensions shown in *Fig. 24* were



'FALL' AT THE VLIETE POLDER, NOORD BEVELAND, 11 SEPTEMBER, 1889.

Mr. R. O. van Manen. reached. In the island of Noord Beveland, where the polders Nieuwe Noord Beveland and Oud Noord Beveland met, there occurred another fall; although this did not cause much ground to be lost, it was an exceedingly dangerous case, as part of the sea-wall of the polder of Oud Noord Beveland was entirely swallowed by the sea. The Author's statement that all canal and inland waters were by law maintained artificially at -0.5 metre (-1.64 foot) A.P. was too general. The hydrography of the Netherlands was very intricate and could not be described in a few lines; it must suffice to say that this level of -0.5 metre A.P. concerned only the North-Sea Canal, which was opened to traffic in 1876. Formerly on the site of the present canal there was an estuary or branch of the Zuider Zee, and when the canal was constructed this submerged area was reclaimed. A level of -0.5 metre A.P. was considered desirable for the North-Sea Canal. Later on it appeared to be difficult to preserve this level continually, even though extensive pumping had been provided for from the beginning. An agreement to maintain the desired level under all circumstances, however, did not exist.

Mr. Saner. Mr. J. A. SANER considered the Paper to be particularly useful in its description of the difficulties met with when protecting land in a low-lying district such as Holland. The nature of the foundations, which appeared to be virtually quicksand, precluded the use of heavy works; and the action of the waves in scouring deep holes in front of the protection-works must be a source of constant anxiety. The examples of the use of reinforced concrete were interesting, but only time could show whether the solid mass was of more use than the multitude of small stones. He feared there would be a tendency for the mass to remain in position and hide from view any undermining until it had assumed dangerous proportions; whereas, in the old method of paving with smaller stones, undermining at once showed itself and claimed attention. In Fig. 6, Plate 1, the system of reinforcement was difficult to follow: in his opinion, the upper rods should also have been continued down to the front of the apron, as, should there be scouring under the toe, the upper part would be in tension and probably break off. In work of this description tension should be provided for either way, as it was quite uncertain on which side the tensile stresses would be. In some remarks on a former Paper by the Author, Mr. Saner had taken exception¹ to the spending of so much money on the protection of the English coast, where the area of

¹ Minutes of Proceedings Inst. C.E., vol. clix, p. 122.

land to be reclaimed was not in proportion to the cost of the works; Mr. Saner. but in a country like Holland the problem was entirely different, and the area reclaimed from the sea was vastly greater in proportion to the length of the embankment. A more parallel case in England would be, for instance, the reclamation of portions of the Wash, supposing such a thing were possible.

Mr. VOISIN, of Boulogne-sur-Mer, remarked that the Author had Mr. Voisin. dealt in a clear and precise manner with a department of the engineer's art in which the Dutch had always shown themselves to be highly skilled, and in which it might well be said that they had created a standard. Recourse had therefore been had to them in past centuries to carry out numerous works of damming, protection, or drainage. In France particularly, Dutch engineers, among others, commenced the first works of reclamation of the marshes of Bordeaux and Médoc, in the seventeenth century, and of coast-defence at Berck, in the middle of the nineteenth century. It was from a study of their methods, therefore, that all, or nearly all, works of a similar nature had been conceived. In the north of France there was a vast district—extending over a large part of the coast of the Département du Pas de Calais, and over all the coast of the Département du Nord—which had been gained little by little from the sea in the course of centuries: this was termed the *Wateringues* region. In the Pas de Calais department this district was divided into eight sections, under administrators who were chosen by election, as in Holland; they administered individually under the control of the State, maintained the drainage-works from their own funds—derived from a special tax—and executed improvements with the co-operation of the State. For the most part, the land, which over an area of nearly 40,000 hectares (98,144 acres) was beneath high-water level, was guarded against the sea by a simple cordon of dunes,¹ but for a length of 935 metres (1,040 yards) it was protected by an important dike of rather antiquated type. Thanks to vigilant supervision, good results had been achieved by timely execution of repairs and strengthening, consisting, so far as the dunes were concerned, of fascine-work or mattresses with a cover of stone pitching. It was the same in the Département du Nord, except that in a zone termed *Moeres* (meaning "marsh," the naming of the whole of the *Wateringues* region being almost entirely of Flemish origin), the land, as in Holland, was beneath low-water level, so that drainage could be effected only by artificial methods.

¹ In certain places this cordon is doubled or tripled by interior dikes, as in the case of the Petten dike, referred to by the Author.—V.

Mr. Voisin. More recently it had been necessary to undertake new defence-works both at Berck, where the embankment had a total length of 800 yards, and 4 or 5 miles north of Boulogne. In these works use had been made of groynes of short projection and very slightly inclined face; of foreshore-protection by means of large rough-hewn stone slabs or fascine-work; and of dikes, of such profile as the situation of the works permitted, faced with fascine-work, concrete, ordinary masonry (blocks or slabs), or, finally, reinforced concrete of the de Muralt type. These latter facings, still fairly recent, seemed likely to yield good results. The general practice was to interpose between the facing and the sand either a watertight layer of beaten marl (*capelin*), or straw, or more often both. Straw had been used for many years in the whole of that district, and it had even been employed for direct facing on some parts of the French coast. Finally, in case of "falls" in any works, recourse was had either to the creation of a berm with a very moderate talus; or to depositing blocks of hard chalk; or to facings consisting of large fascine-mattresses—a system which so far had yielded excellent results. Mattresses of the de Muralt type had not yet been employed in the north of France. It would thus be seen that the technical experience of the Dutch engineers had been put to great profit in France.

Mr. Wisselink. Mr. W. J. WISSELINK, of Amsterdam, observed that the Author had mentioned only the objections to Mr. Sanders's scheme for a dam across the Zuider Zee; and, at that gentleman's request, he wished to state the answers to these objections.¹ The bottom of the Zuider Zee on the site of the enclosing dam consisted of a thick layer of sand, and in Holland even church-towers were built (without any other foundation) on such a support, the pressure allowed reaching as high as 3 to 5 kilogrammes per square centimetre (2·9 to 4·8 tons per square foot). The proposed sea-wall would cause a pressure of only 1 kilogramme per square centimetre on the sea-bottom; consequently the risk of settlement—the first objection—would be very small. Under water the sea-wall itself would be composed of *geplempt* sand, that was, sand tipped into the water, and above water of *opgespoten* sand, or sand pumped from the bottom by hydraulic means. Every practical engineer knew that the risk of settlement of such a wall might be neglected. A very simple graphical calculation would show that the cylinders, even when not filled with sand, possessed sufficient stability to resist overturning. Scouring from beneath by the sea would be possible

¹ These are set forth fully on pp. 70-75 of the work by Mr. Sanders referred to on p. 19.—W. J. W.

only if the tides through the Tesselschegat at Harlingen were to sweep round in a curve, the current washing against the north side of the dam between Wieringen and Piaam; the Government Committee rather expected the contrary to happen, and prophesied accretion of sand along the entire north side. Scouring between the cylinders during construction could easily be obviated by placing "dam-plates" there, projecting a foot or two above the bottom; the current would be guided by these so as to prevent scouring. For closing the last great opening left in the dam, the *zinkstukken* referred to on p. 12 might be used on both sides of the range of cylinders.

The AUTHOR, in reply, considered himself fortunate in having evoked so wide a discussion, especially by many gentlemen holding high official positions in Holland. With regard to the comparative profiles of the Dutch and Essex sea-walls, the Essex walls were to a large extent protected by vast sandbanks from exposure to the sea. As to the temperature of the subsoil, provided land was well drained, there was comparatively little variation as compared with the temperature of the air. The questions of the rotation of crops and of agricultural limitations in Holland generally were rather beyond the scope of the Paper. The extent of the percolation in the sand-dunes had been determined by observation. The planting of trees on the leeward side of the sand-dunes was an expedient that had been adopted on many dunal coast-lines. The effect of sand blowing over the crest of the dunes was doubtless to destroy many of the trees planted on the leeward side; but even though these trees perished, they fixed the sand slopes in the rear, and arrested the landward movement of the entire mass of the dune. It was for this reason that in many spots vegetation varying between a light scrub and small forest-trees existed on slopes lying at a very steep angle on the landward side of dunes. Mr. Bell had interpreted rather too literally the Author's statement about the flocks following the tide. What was meant was that as rough herbage sprang up on saltings in advance of marshes, when these slowly became reclaimed land, the flocks gradually took possession of such land and helped to render it of value. The legal steps antecedent to the reclamation of alluvial lands in Holland were regulated by a law of 1904. Private owners carrying out reclamation of lands contiguous to their own property did so at their own cost and the land became their own. When alluvial Crown land was to be reclaimed by private individuals, application for a concession had to be made to the State. Caution-money for the due execution of the works had to be lodged. The individual

The Author. who desired to embank such land had to make all arrangements as to survey, etc., and the reclaimed land became the joint property of the State and the private owner. An instance of such procedure was a recent reclamation carried out to the north of the Slaakdam, which was the joint property of the State and the proprietor of the St. Philipsland estate. These conditions were regulated by a law of 1900 on Embankment and Drainage. The material required to form the embankment was taken, as in Essex, from the marshes on the landward side. The question raised by Mr. Bell, as to the source of such material, should the embankment of the Zuider Zee ever be carried out, would doubtless be an important point when the details of the work were taken in hand. How the construction of reclamation-dams was first commenced was an unsolved question, not only in Holland but also in the fen-lands of Great Britain, which might well form a subject for exhaustive discussion at the Institution. It was generally believed that the embankment of the marshes in the estuary of the Thames was Roman handiwork. There were districts in Essex where the character of the clay of which the embankments were composed seemed to indicate that it had been brought by water from another locality, as it was different from any now existing in the neighbourhood. Doubtless "race-pressure" had been the principal origin of primitive reclamations. Professor Butler's description of the winning of land on the Canadian coast was very interesting. The formation of estuarial land by means of "running dikes" produced conditions very similar to those existing on the Caldicote Levels, Monmouthshire, where also the tidal range was extreme, attaining 52 feet. The Caldicote Levels were, however, maintained by higher defences than those described by Professor Butler—in some cases as high as 45 feet.

He was of opinion that the de Muralt systems were eminently adapted for use above low-water level in England. In several localities the lighter type was already adopted, and the use of the heavier type was, he thought, a question of the immediate future. He certainly believed that the Dutch system would eventually supersede the present type of work on most new installations. With regard to the de Muralt mattress system, he had under consideration its use in the estuary of the Thames at a spot where, owing to the severity of the tidal current, the expense of laying down random stone year by year, in order to check dangerous scour, was extreme. He understood Mr. Cruttwell to suggest that between a clay embankment and its stone pitching a pervious material, such as gravel, might be interposed. The present system of stone pitching produced the effect described by Mr.

Cruttwell, and every year, on exposed portions of a level, orders The Author. had to be made for "keying up," which meant the readjustment and tightening of the pitching. The Author thought that this effect would be rather accentuated by the use of gravel between the pitching and the clay embankment. The chalk backing generally used with stone pitching was, of course, pervious. Concrete slabs laid directly on a clay embankment, as a veneer, would probably give the best results. Mr. Haupt's view appeared to be that the use of groynes might generally be dispensed with. In this, however, the Author could not concur, as he believed that in almost all localities threatened with erosion it was in the direction of groyning that successful protection lay. Of course, this rule was not universal. There were few problems so delicate as the maintenance of the balance of equilibrium on a threatened sea-frontage. In a number of instances which had come under the Author's notice so-called defence-works had merely accentuated the conditions they were intended to cure. This, however, did not obscure the fact that it was by a well-devised application of groynes that the movements of littoral drift could be arrested and rendered protective. The two jetties shown in *Fig. 20*, p. 57, were certainly very unfortunate expedients. The Author did not regard as a scientific device the system of slope-protection by means of concrete blocks strung on wires, mentioned by Mr. Latham. It might perhaps occasionally be of service on flat slopes of sheltered river-banks: if used on an exposed foreshore it would probably not last long. In any event it perished when the wire gave way from corrosion or other cause. He agreed with Mr. Saner that in the system of reinforcement shown in *Fig. 6*, Plate 1, the rods should be continued down the foot of the apron. Mr. van Gelderen's remarks on the protection of the Island of Walcheren did not conflict with the statements contained in the Paper. As to Mr. de Groot's explanatory notes on the construction of groynes below low water, the Author could not pit his experience against Mr. de Groot's long practice of protective work on the Dutch coast. He might point out, however, that the construction of mass concrete work under water was not uncommon in breakwater foundations, which bore a close analogy to the seaward extensions of Dutch groynes. According to the Author's experience such concrete must not be of less strength than 5 to 1. He agreed that some system such as that of fascine work was essential wherever anything approaching a quicksand foundation existed.

Turning to Mr. Blocq van Kuffeler's very interesting remarks, the Author desired to call attention to a Paper of high value, by the distinguished Dutch geologist, Professor G. A. F. Molengraaff,

The Author on "The Subsidence of the Soil of the Netherlands."¹ This exhaustive Paper, dated June, 1909, might perhaps be regarded as the last word on the subject of land oscillation in the Netherlands. From the fact that a substratum of peat, which existed almost everywhere in the provinces of South Holland and North Holland, was overlain by blue sea-clay, generally at a level of -12 metres and in some spots at -19 metres A.P., Dr. Molengraaff argued that the northern and western portions of the Netherlands had been subject to subsidence. The eastern and south-eastern portions of the country were little affected by this action. The opinion was expressed that the interior of Holland was thus largely converted into a lagoon or *Haff* in the inter-glacial period which succeeded the great glacial epoch and preceded the last period of extension of the pleistocene glaciers. The accumulation of sediment brought down by the Central-European river-system sporadically counterbalanced the effect of depression. Dr. Molengraaff stated that at the beginning of the recent period the dunes extended far seaward of their present site and that the disturbance of the sea which brought into being the Straits of Dover also largely destroyed the dunes. The lagoons had then a depth of about 6 metres. This period of depression was followed by upheaval, and the latter again by subsidence. Dr. Molengraaff also stated that, within the historic period, sinking of the land had continued, the loss of land being accelerated by the rapid wearing-away of the dunes on the sea-side and more inland by floods caused by storms. The estimate of the subsidence which had occurred during historic times varied between 25 and 90 centimetres (10 and 36 inches) per century. Professor H. G. van de Sande Bakhuyzen, from observations of the level of A.P. at Amsterdam,² concluded that between 1700 and 1860 no change of land-level occurred at Amsterdam. Mr. J. C. Ramaer,³ from observations on gauges taken during the years 1862-1906, thought he had established the fact that the soil of Holland was now sinking at the rate of 18 centimetres ($7\frac{1}{2}$ inches) per century. The Author had noted on portions of the Essex coast-line conditions which appeared to point to the probability of similar subsidence on this side of the North Sea. Mr. van Kuffeler's remarks on the discharge of the polder water did not differ from

¹ Koninklijke Akademie van Wetenschappente Amsterdam, vol. xii, p. 468.

² "The height of the mean sea-level in the Y before Amsterdam from 1700 to 1860." *Ibid.*, vol. x (1908), p. 703.

³ "De daling van den bodem van Nederland," etc. *Tijdschrift van het Koninklijk Instituut van Ingenieurs, Verhandelingen*, 1907-8, p. 1.

those contained in the Paper. The investigation carried out by The Author Mr. Pennink, and referred to by the Author, had reference to the question of water-supply; the compensating reservoirs were a drainage matter. The essence of Mr. Pennink's warning was, however, that every precaution must be taken against piercing the salt-water substratum under the town of Amsterdam, and thus causing fusion of the zones of fresh water and salt water underlying that city. The Author had stated 33-66 feet, on Mr. Pennink's authority, as the depth of the surface of the brackish water below the intake. The planting of dunes with helm-grass was, of course, largely practised on the English coast. The roots of the grass passed vertically downwards for a considerable distance, and then spread laterally. It was obviously desirable to make the slope at the foot of dunes as flat as possible. The work carried out at Renesse (Fig. 9, Plate 1) had impressed the Author very much, as the blown sand was cleverly trapped in order to restore the contour of the threatened dune. The safety of the neighbouring lighthouse would now appear to be assured. As to the Zuider Zee, the Paper described Mr. Sanders's proposal in some detail. Mr. Sanders had suggested that the de Muralt system should be applied for the protection of both sides of the embankment; and, as stated by Mr. Wisselink, the scheme bore evidence of boldness of conception and care in elaboration. British engineers regarded with the deepest interest the grand project of the reclamation of the Zuider Zee, and looked forward to seeing this work taken in hand seriously. Mr. van Manen's further information on the subject of "falls" was extremely valuable. On the subject of the level of canals and inland waters in Holland, the Author gathered that an artificial level had to be maintained, but this level was not to one standard throughout. Mr. van Manen did not, however, state the extent of the variation.

10 January, 1911.

ALEXANDER SIEMENS, President,
in the Chair.

The PRESIDENT mentioned that, since the members last met, The Institution had lost a very old member, Sir John Aird having died after a long illness. The Council had passed the following resolution, which they had no doubt the members would support :—
“The Council have learned with sorrow of the death of Sir John Aird, who has been an Associate of The Institution for 51 years, having served for several years as an Associate of Council; and they desire to convey to the members of his family an expression of deep regret and sympathy.”

The Council reported that they had recently transferred to the class of

Members.

EDWARD VINCENT ACTON.	DAVID WILSON HOOD.
JOHN BALL, Ph.D. (<i>Zürich</i>), D.Sc. (<i>Lond.</i>)	HENRY ADIN HULL.
JOHN JOB CREW BRADFIELD, M.E. (<i>Sydney</i>).	TIEN YOW JEME, Ph.B. (<i>Yale</i>).
GEORGE HERBERT BROOME.	CHARLES JAMES JENKIN.
JOSEPH WILLIAM BUCKLEY.	WILLIAM THOMAS LANCASHIRE.
THOMAS HOWARD CHAPMAN.	KENNETH SMALE LAURIE.
GEORGE FREDERICK CHARNOCK.	JOHN GEORGE MORLEY.
HENRY HARVEY DARR, M.E. (<i>Sydney</i>).	HENRY VEREKER LLOYD MORONY, B.E. (<i>Queen's</i>).
WILLIAM STEWART HOBART DORMAN, B.A., B.E. (<i>Royal</i>).	JOSEPH LOYS NAZ.
ALFRED DRYLAND.	GEORGE HOLT ORMEROD.
HARRY HORNE ELLIOTT.	ARTHUR HENRY OUGH.
ROBERT ELLIOTT, B.Sc. (<i>Glas.</i>)	HENRY LAWRENCE PEARSON.
FREDERICK AUSTEN HADOW.	VINCENT LITCHFIELD RAVEN.
CLEMENT DANIEL MAGGS HINDLEY, M.A. (<i>Cantab.</i>)	HENRY JOHN RUDGARD.
	WILLIAM PARK WEIR.

And had admitted as

Students.

HAROLD WILLIAM ASHEY.	ALFRED TRAVERS FAIRTLOUGH SMITH,
JOHN WARDEN EDWARDS.	B.Sc. (Engineering) (<i>Lond.</i>)
FELIX JOHN HOOKHAM.	ERIC OWEN STUBBINGS, B.Sc. (Engi-
EUSTACE ERIC DONALD McDONALD.	neering) (<i>Lond.</i>)
FRANCIS HENRY MARTIN, B.A. (<i>Cantab.</i>)	

The Scrutineers reported that the following Candidates had been duly elected as

Members.

WILLIAM COATES REYNOLDSON. | ROBERT LLOYD ROBERTS.

Associate Members.

CHARLES DUDLEY ARNOTT, Stud. Inst. C.E.	KENNETH LIGHTFOOT.
CHARLES FOSTER ATCHISON, Stud. Inst. C.E.	JOHN NAPIER LIST, M.A. (<i>Cantab.</i>)
JOSEPH FREDERICK BRENNAN.	ALFRED TUDOR MACDERMOTT, M.A., B.A.I. (<i>Dubl.</i>)
GEOFFREY CHASEMORE BURNAND, B.A. (<i>Cantab.</i>)	DUNCAN NORMAN MACPHERSON.
WILLIAM DONALD McDONALD CRUICKSHANK.	FRANCIS JOSEPH MILLS, Stud. Inst. C.E.
ANDREW BAINBRIDGE FERGUSON.	WALTER LANCELOT MOORE.
SIDNEY CHARLES GLADWYN, Stud. Inst. C.E.	PHILIP FREDERICK MURPHY, Stud. Inst. C.E.
WALTER JOSOLYNE GLANVILLE, B.A. (<i>Cantab.</i>)	ALEXANDER McCULLOCH PATERSON.
FRANCIS CHARLES HAY.	GEORGE MUSKETT SAUNDERS.
EDWYN JERVOISE.	THOMAS ALFRED SHAW, Stud. Inst. C.E.
FREDERICK TEMPLETON MANHEIM KISSER, B.Sc. (<i>New Zealand</i>).	ARTHUR MACKLOW SMITH, B.Sc. (Engineering) (<i>Lond.</i>)
ONWARD BAYES LACEY.	FRANK WOOLLASTON TIMMIS.
RALPH WHITTINGTON LONDON, Stud. Inst. C.E.	HARRY KRAMER TRECHMANN.
PERCY HERBERT LEE.	ROBERT WILLIAM WATSON.
	LAWRENCE TRENCH WILSON, Stud. Inst. C.E.
	SIDNEY GEORGE WINN, Stud. Inst. C.E.

Associates.

HECTOR HAROLD BOND. | JAMES WILLIAM HORNE, B.Sc. (*Edin.*)

(*Paper No. 3876.*)

**"The Strengthening of the Roof of New Street Station,
Birmingham."**

By WILLIAM DAWSON, M. Inst. C.E.

THIS roof, erected in or about the year 1852 over the old portion of New Street Station, has a span ranging from 191 feet at the east or London end to 212 feet at the west or Crewe end.

Its construction has been described in the Proceedings,¹ and at the time of its erection its span was referred to as "the largest hitherto attempted," the nearest to it being the roof over Lime Street Station, Liverpool, of 150 feet span. The latter, however, was replaced by a roof of 220 feet span in 1871. Both roofs belong to and are maintained by the London and North Western Railway Company.

Each principal of the Birmingham roof, of which there are thirty-six, is of the bowstring girder type, consisting of a rectangular arched rib, a tie-rod 4 inches in diameter, and vertical members with intermediate cross bracing (Fig. 1, Plate 3). The rib is splayed out at the bearings with cast-iron distance-pieces, giving a bearing-surface of 20 inches by 15 inches.

The principals are 24 feet apart from centre to centre (Fig. 2, Plate 3), with one end resting on the station-walls and the other on cast-iron columns. The height from rail-level to the crown of the arched rib (exclusive of the lantern or jack-roof) is 75 feet. The arch has a rise of 42 feet above the springing, and the tie-rod a versed sine

¹ J. Philips, "Description of the Iron Roof, in one Span, over the Joint Railway Station, New Street, Birmingham." Minutes of Proceedings Inst. C.E., vol. xiv, p. 251.

of 18 feet. It was erected by means of travelling stages from the platform, Messrs. Fox, Henderson and Company, of Smethwick, being the contractors. The covering consisted partly of corrugated iron and partly of glass, but owing to corrosion the former was replaced by zinc on spars, as shown in Figs. 1, Plate 3, the alteration being completed in 1873.

NECESSITY FOR STRENGTHENING.

This roof is practically of the same design and construction as the one over Charing Cross Station, which was erected about the year 1863 and removed early in 1906; indeed, it may be taken as the prototype of that roof. The span of the Charing Cross roof was 48 feet less than the Birmingham roof, namely, 164 feet as against 212 feet, but the principals were farther apart, ranging from 35 feet to 37 feet between centres as against 24 feet at Birmingham.

In view of the failure of one of the tie-rods of the roof at Charing Cross it was deemed advisable to subject the welded joints in the tie-rods at Birmingham to a close scrutiny under a magnifying glass, after they had been thoroughly cleaned. No serious defects were discovered, but having regard to the results of the inquiry into the Charing Cross disaster, when it was proved that an original bad flaw in the weld of a bar could remain undetected in spite of the fact that every examination and test of the ironwork was made;¹ and in view of the length of time the roof had been subjected to the deleterious effect of steam and fumes from the locomotives, together with the fact that it was of rather lighter construction than accorded with modern practice, Mr. E. B. Thornhill, then Chief Engineer, advised the Railway Company to provide auxiliary tie-bars.

It was found by calculation that the tensile stress in the main tie-rod was 4·37 tons per square inch for the dead load alone, or 5·84 tons if the weight of the painters' scaffolding and 6 inches of snow were added; and that for each 10 lbs. per square foot of horizontal wind-pressure, the stress would be increased by an additional 0·42 ton per square inch. Therefore, for a wind-pressure of 40 lbs. and other loads as stated, there would be a stress in the tie-rod of 7·52 tons per square inch; but as it is unlikely that 6 inches of snow will remain on the roof with a wind blowing

¹ Major Pringle's Report to the Board of Trade, 25th March, 1906. See "Returns of Railway Accidents and Casualties . . . during the Three Months ending the 31st March, 1906," p. 122. London, 1906.

at a velocity corresponding with a horizontal pressure of 40 lbs. per square foot, 0·59 ton should be deducted, making a net unit stress of 6·93 tons.

It was also found that the compressive stress in the main arch-rib would be 2·75 tons per square inch for dead load plus painters' scaffolding, and 10 lbs. horizontal wind-pressure; or 3·17 tons per square inch with 40 lbs. wind-pressure. As the rib was well braced with purlins, wind-ties, and roof-covering, it was decided that the strengthening of the tie-rod alone need be dealt with.

SCAFFOLDING.

The problem that had to be solved was, how to fix an auxiliary tie-bar, calculated to weigh $5\frac{3}{4}$ tons, without interfering with the enormous traffic that day and night was to be found on the platforms below. To erect travelling stages, as had been done when the roof was originally constructed, was out of the question, and ultimately it was decided to suspend scaffolding from the existing principals and to strengthen them temporarily in such a way as to put no additional stress on the tie-rods.

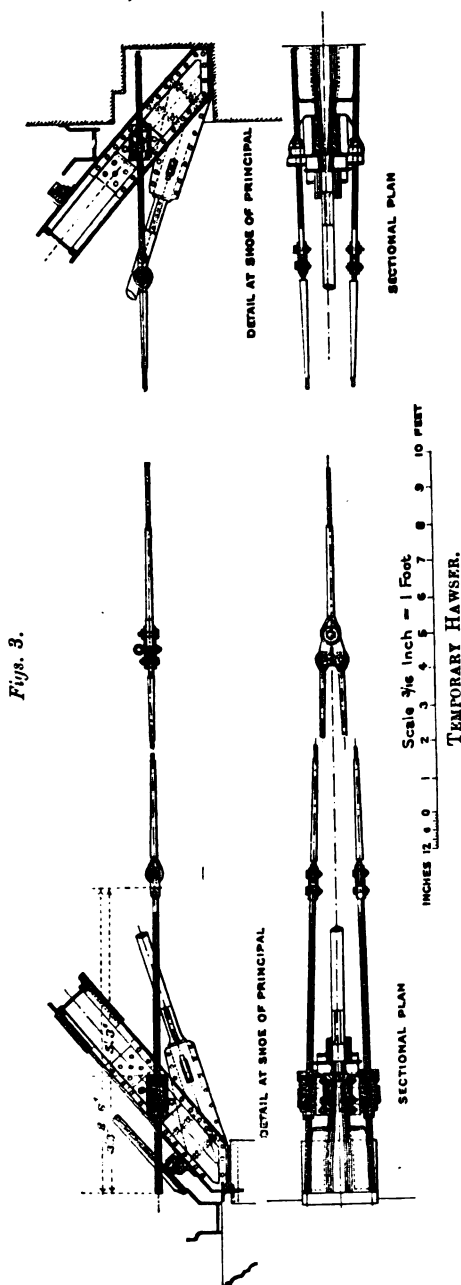
The scaffolding adopted, shown in Figs. 4 and 5, Plate 3, involved placing a load of 24 tons on each principal, which would have produced an additional stress of $26\frac{1}{2}$ tons on the existing tie-rod. In order to avoid this additional stress, a steel-wire hawser with a breaking strength of $63\frac{1}{2}$ tons was attached to the feet of each principal during the whole of the time scaffolding was suspended from that principal. The hawsers were provided at each end with long screws, which at the column end passed through springs securely nested in a channel-bar bracket bolted to the shoes of the principals, as shown in *Figs. 3*. The length of the screws allowed for adjustments to suit the varying spans of the principals.

The springs had been previously loaded in a testing-machine, when their lengths between two gauge-points under the required load was accurately noted. The tension required in the hawser to counterbalance the effect of the scaffolding on the tie-rods was 16 tons, and to determine when this tension was on the hawser it was only necessary to screw down the nuts on the hawser-screws until the springs were compressed to the required extent. The adjacent end of the roof-principal was lifted clear of its bearing during the time the hawser was adjusted, and lowered after the adjustment had been completed. The erection of the scaffolding was then proceeded with, working from each end of the principal towards the centre, so as to avoid unequal loading. Three hawsers were in use with each

set of scaffolding, one hawser being lowered and refixed to the third principal in advance during an interval of an hour between trains on successive Sundays.

The general arrangement of the scaffolding is shown in Figs. 4, Plate 3. It will be seen from the Section A A that sufficient scaffolding was provided to enable the work of fixing the tie-bars to be continued without interruption. Thus, while the scaffolding required for fixing the tie-bar to one principal was in use, the other set of scaffolding was being moved forward from the principal in the rear to the one in advance of the work. Runners between adjacent principals and along a principal were fixed as shown, to facilitate this work; and altogether the scaffolding was found to answer its purpose very satisfactorily.

Each lower scaffold or gangway was made of twelve similar sections for each principal, suspended direct from the main rib and guyed from adjacent principals by wire ropes. All the



parts of one section were interchangeable with those of any other section. The lengths of the different units of the steelwork supporting the gangway were made to suit the shortest-span principal that it was intended to work under, and a series of connecting bolts was fixed between the different units. These connecting or adjusting bolts were provided with long screws and double nuts, so that the gangway could be adjusted to the necessary length and made perfectly rigid throughout its entire length by merely tightening up the adjusting bolts as required. When the tie-bar was completed, this gangway was readily stripped, hoisted to the stage at the level of the tie-rods, and refixed under the next principal but one in advance, no scaffolding being lowered to the platform until the whole of the work had been completed.

A gang of special scaffolders was continuously employed with each set, and, as they had previously been accustomed to this roof, they were able from the first to work with confidence at so great a height from the ground, and to remove and refix about 50 tons of material per week without mishap or inconvenience to any of the passengers using the platforms below.

Canvas sheets, commonly used in painting large roofs, were suspended below the whole area where work was in progress.

ERECTION OF NEW TIE-BARS.

The new tie-bars consist of two flat steel plates on edge, each measuring 6 inches by $\frac{3}{4}$ inch, with a space of $1\frac{1}{2}$ inch between them for painting purposes, and they are fixed about midway between the old tie-rod and the level of the springing-line. Steel rods, $1\frac{1}{8}$ inch in diameter, suspend the bar at each of the vertical struts of the old roof, one rod on each side of the principal hanging from brackets fixed on either side of the arched rib. Full particulars of this arrangement are shown in Figs. 6, Plate 3. Each suspension-rod was provided with screwed ends so that the nuts could be used to adjust either the rod itself or the flat bars of the new main tie in any required way.

Before any material for the tie-bars could be ordered, it was necessary to ascertain the length of each individual tie-bar, since, owing to the variation in the span and rise of the roof and to local irregularities in the principals themselves, no two lengths of bar were exactly the same. Two lines parallel to the north wall of the station and 130 feet apart were very accurately set out on the platforms on two successive Sundays, when the station was specially

closed for a few hours for that purpose. Heavy plummets were then suspended from each arch-rib in succession vertically over these lines, and other plummets were similarly suspended at each vertical of the principal, and the distance between them was accurately measured and checked with reference to the main plumb-lines.

The rise of the tie-bar having been determined, the length of the individual bars could then be calculated. This method proved efficient, and in no case was any alteration of a bar necessary. The vertical rods were delivered of the approximate length required, and were cut and screwed on the ground as the work proceeded. The tie-bars were delivered with cover-plates riveted on one end and with the holes drilled in the cover-plates for the end of the adjacent length in which no holes were provided. The whole tie-bar was then adjusted to the correct camber by means of the vertical suspension-rods, and the holes were drilled through, using the cover-plates as templates. The joints were riveted up by pneumatic riveting-hammers.

The method of attachment of the new tie-bars to the old principals required special consideration, owing to the complex nature of the ironwork where the old tie-rods intersected the foot of the arched rib. It was eventually decided to bore a hole through the entire thickness of the web-plates and the cast-iron packings forming the splayed portions of the main rib, and to insert tightly fitting steel pins $2\frac{3}{4}$ inches in diameter.

Castings were made and fitted to the shoes of each principal, to provide support for the pins between the web-plates of the roof-principal and the new tie-bar.

In view of the importance of the pins being absolutely level and at right angles to the new tie-bars, plant was devised to enable the holes to be bored with great accuracy. A hole $2\frac{1}{4}$ inches in diameter and 10 inches long was first drilled through the shoe of the principal by a Whitelaw pneumatic drilling-machine, and the castings, which were delivered with a $2\frac{1}{4}$ -inch diameter hole already drilled through them, were then fitted in place and secured to the principal by three 1-inch bolts. A boring-rig, consisting of a modification of an ordinary side-light cutter attached to a face-plate, was then secured to the principal. The boring-bar passing through the $2\frac{1}{4}$ -inch hole was housed in a bearing attached to a similar face-plate on the opposite side of the principal, and was driven by an ordinary pneumatic drill through worm-gear. After the boring-bar had been carefully adjusted, the hole—passing through 10 inches of old metal and 12 inches of new, i.e., 22 inches in all—was bored out to a plug-gauge $2\frac{3}{4}$ inches in diameter, with four to six passes

of the boring-bar. The pins, provided with pilot-nuts, were forced into the holes by means of jacks.

On opening out the shoes of the principals on the north wall of the station, it was found that the main wall of the hotel was carried by brick arches, and boring the holes for the main pins through the shoes of the roof-principals involved cutting away a considerable portion of the haunches of these arches. This was a difficult and costly operation, necessitating modifications of the boring-plant, and requiring the utmost care to prevent any disturbance in the main wall of the Queen's Hotel above.

The work in the hotel smoke-room and corridor also proved difficult. Owing to the confined space available and to the close proximity of the steel framing of the smoke-room to the roof-principals, the fixing of the new tie-bars involved making special arrangements for obtaining access to each of the seven principals; further, owing to the necessity of avoiding complaints from visitors at the hotel, the work could be carried on only between Friday night and Monday morning.

When detailed particulars of the principals resting on the garage were obtained, it was found necessary to provide angle-shaped girders to transfer the stress from the new tie-bars to the foot of the principal (Figs. 7, Plate 3). These girders weighed $2\frac{1}{2}$ tons each, and the feet of the old principals had to be underpinned and propped during the time the tie-bars were being fixed.

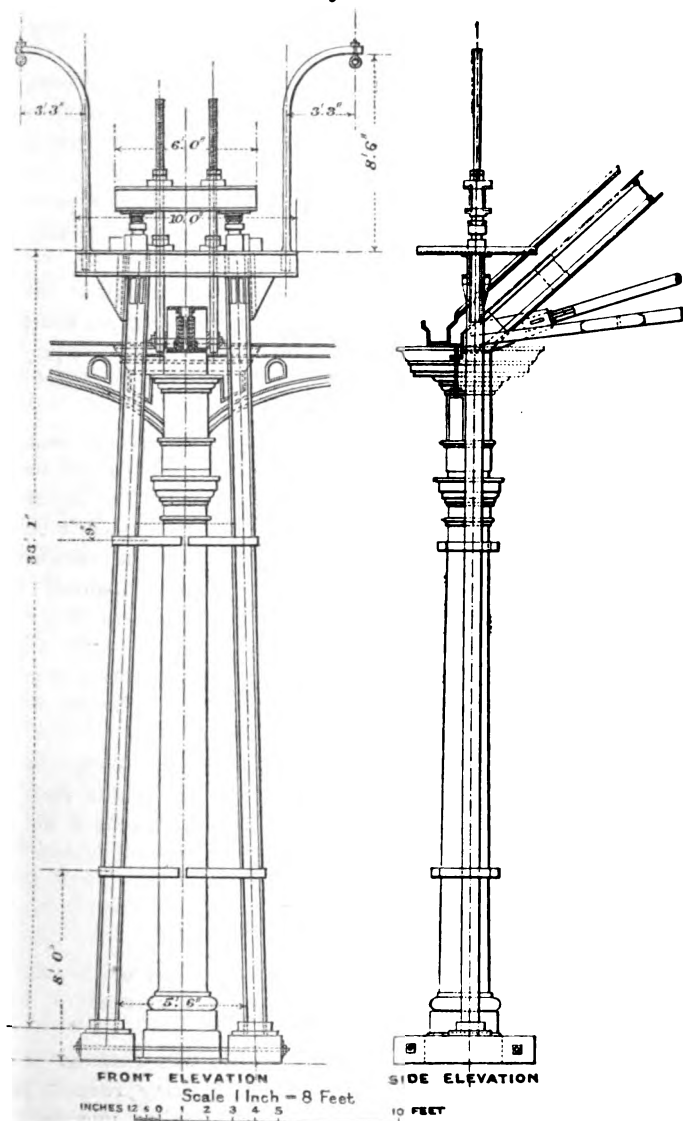
STRESSING THE NEW TIE-BARS.

In strengthening a roof or any engineering structure by the addition of auxiliary members, it is of course very important that initial stress shall be put upon the latter, or in other words, that the old members shall be relieved of a certain amount of the stress; and where the amount can be determined accurately this procedure makes the work much more satisfactory and efficient. The method adopted in the present case for ascertaining accurately the amount of stress put upon the new bars was the employment of calibrated springs, secured temporarily to the new tie-bars in the manner shown in Figs. 6, Plate 3.

A hole $2\frac{1}{4}$ inches in diameter was drilled through the column end of the tie-bar, 6 feet from the main pin, and a pin was inserted to take the ends of two adjusting-screws, which passed through the springs secured to the adjoining lengths of tie-bars. One inside plate and two outside cover-plates were attached to the column end of the new tie-bars, between which plates the two ends

of the adjoining length of tie-bar were inserted, the cover-plates

Figs. 8.



DERRICK FOR LIFTING END OF PRINCIPAL.

acting as guides when the bars were drawn together by compressing

the springs. The springs had been carefully calibrated under a load of 12 tons each. The adjusting-screws were eventually tightened until this load was put upon them and thereby transferred to the new tie-bar, the ends of which were drawn together to the extent of $\frac{7}{8}$ inch.

Simultaneously with the operation of putting stress upon the new tie-bars, the hawser-screws were tightened, so as to maintain the full tension of 16 tons in the hawser, and this stress, with the stress of 24 tons in the tie-bars, was continued while holes were drilled in the end of the tie-bar and the joint was riveted up and allowed to cool; the amount of stress in the new tie-bars was thus definitely assured.

The adjusting-gear on the tie-bar was then dismantled, but the hawser remained until the scaffolding on the principal had been removed. The springs were tested periodically during the progress of the work, but no alteration in their gauge-length under load was observed.

While the adjustment was in progress, the end of the principal was lifted entirely off its bearing by a specially-constructed derrick provided with long rods forming a sling (*Figs. 8*). The object was to ensure that none of the stress put upon the springs could be transferred to the columns in any way, otherwise the efficiency of the whole operation would have been seriously impaired. The adjustment, therefore, was made with the principal swinging clear of its bearing. Moreover, there would have been some risk to the column itself had not the weight of the principal been removed before the adjustment was made. The weight lifted by the derrick was 42 tons at a level of 35 feet above the ground.

Roller-bearings had been provided in the original design of the roof, but after lifting the principals it was discovered that the rollers were completely rusted up and were of no value for the purpose intended. They were therefore removed and replaced by solid packings.

EFFECT OF ALTERATION.

As was anticipated, the effect of shortening the old tie-rod by relieving it of some portion of its stress was to cause a slight buckling of the cross bracing in the principals.

The initial tension put upon the new tie-bars resulted in the stress on the tie-rods due to dead load only being reduced from 4.37 tons to 2.08 tons per square inch. It is uncertain to what extent stress due to snow, wind-pressure, painters' staging, etc., will be divided between the old tie-rod and the new bar.

Taking the extreme view of assuming that the old rod takes the whole of these extraneous loads and that the roof is subject to a wind-pressure of 40 lbs. per square foot, the maximum stress that could then be brought upon it will be 4·27 tons per square inch. In other words, the unit stress in the old tie-rod will vary between a minimum of 2·08 and a maximum of something less than 4·27 tons per square inch; whilst in the event of failure of one of the old rods, the new tie-bar would safely carry the whole of the load.

Owing to the greater depth of the principal, the effect of the new tie-bar is to reduce the compression in the arched rib; and as the stress was originally not excessive, the question does not call for further consideration.

The effect of the weight of the painters' scaffolding on these large-span roofs, especially when the distance between the principals is considerable, is perhaps of greater importance than is sometimes realized. The ordinary painters' scaffold for maintenance purposes in this case weighs 8·12 tons for each bay of roofing, and the additional stress in the tie-rod due to this scaffolding is 0·88 ton per square inch. Further, the tie-rods themselves may be strained locally by having to bear the bending-moments caused by the scaffolding, as well as the stress due to their position in the principal. In the Birmingham roof it is arranged that the scaffolding shall in all cases bear upon the parts of the tie-rod immediately adjoining the coupling-boxes or points of support of the rod, so as to avoid these additional bending-moments as far as possible.

The old tie-rods were subject to considerable vibration, especially when the painters' scaffolding was being moved along the roof from one bay to another. In order to obviate this, two longitudinal tie-rods have been fixed from end to end of the roof, as shown at B B in Figs. 6, Plate 3. Each of these tie-rods is provided with a collar bolted around the old tie-bar at one-third points in the span of the roof, and is also provided with a coupling-box in the centre of each bay, so that any required adjustment can be made to keep the old roof-principal strictly in line.

GENERAL.

Two sets of pneumatic plant, worked at 100 lbs. per square inch pressure, were used for drilling and riveting, compressed air being conveyed over the roof by pipes in the usual manner. The engines and boilers were fixed in a convenient place below.

Artificial light was provided by means of a number of 50 candle-power acetylene lamps. A 1,000 candle-power light was also tried

for a time, but the shadows cast by it rendered its use on the scaffolding dangerous.

The work throughout was so arranged that two new tie-bars could be fixed simultaneously. Principals Nos. 18 and 35, shown in Fig. 2, Plate 3, were commenced at the same time, and the work was carried on from these two points in an easterly direction, until the whole of the work was completed at the rate of two principals per week. The first two principals were finished on the 28th October, 1906.

The steel was made by the Siemens-Martin process, and was specified to have an ultimate tensile strength of 28 to 32 tons per square inch, an extension in 10 inches of not less than 20 per cent., and a contraction of area at the point of fracture of not less than 40 per cent.

In addition to the usual tensile tests, one of the main pins was tested in single shear at each end by a shear-blade with a cutting-angle of 90° . The $2\frac{3}{4}$ -inch pin was cut down to an area of 5.92 square inches, and the results of the tests were as follows:—

Maximum shearing load at end—

(i) 129.31 tons = 21.84 tons per square inch.

(ii) 118.1 " = 19.8 " " " "

The tensile test of the material from which the pin was made gave the following results:—

	Diameter of Test-Piece.	Breaking Weight.	Elongation in 10 Inches.	Contraction of Area.
	Inch.	Tons per Square Inch.	Per Cent.	Per Cent.
1st test	0.81	30.8	23	45.2
2nd „	0.94	31.3	22	46.1

The steelwork was made by Messrs. Cochrane and Company of Dudley, to dimensions and patterns supplied by the Railway Company. The whole of the work on site was carried out by the Railway Company under the direction of the Author, acting for the Chief Engineer, who at that time was Mr. E. B. Thornhill, M. Inst. C.E. The Author's assistant, Mr. W. J. Fuller, was the Resident Engineer.

The Paper is accompanied by eight drawings, from which Plate 3 and the Figures in the text have been prepared, and by a number of photographs.

(Paper No. 3877.)

“The Reconstruction and Widening of Arpley Bridge, Warrington.”

By WILLIAM DAWSON, M. Inst. C.E.

THE London and North Western Railway Company's line between Liverpool and Garston and the Manchester and Yorkshire districts, crosses the River Mersey immediately to the east of Arpley Station, Warrington, by a girder-bridge of 180 feet clear span. The bridge, which, in addition to passenger-traffic, carries heavy goods- and coal-traffic, was erected in 1853 from designs by the late Mr. John Lister, the girders being built on a timber stage supported by piles driven into the river-bed. The line was constructed under the Warrington and Altrincham Junction Railway Act of 1851, and opened in 1853. It was afterwards acquired by the London and North Western Railway Company.

Before the recent alteration which is the subject of this Paper, the superstructure of the bridge consisted of a centre box girder 190 feet long and 16 feet 9 inches deep, and two outer or face-girders of the single-plate web type each 13 feet deep (Figs. 1 and 2, Plate 4). A close timber flooring 9 inches thick was suspended by bolts from the bottom flanges of the three main girders, and to this floor the permanent way was fastened directly.

A footway 5 feet in width was provided on each side of the bridge, supported on wrought-iron cantilevers attached to the outside of the face-girders.

The main girders were of wrought iron with the exception of the top flanges, which were partly of cast iron and partly of wrought iron, the cast-iron portion of the flange consisting of a series of segments in 8-foot lengths with riveted joints. The castings were of 2½-inch metal, and were riveted to the wrought-iron portion of the flange. The bottom flanges were of unusual width, being

6 feet wide in the centre girder and 4 feet wide in the outer girders. The web-plates ranged in thickness from $\frac{1}{2}$ inch at the ends of the girders to $\frac{3}{8}$ inch at the centre.

Overhead bracing in the form of small, curved, box girders was provided at each end of the bridge and at two intermediate points.

SCHEMES FOR RECONSTRUCTION.

The increase in the weight of locomotives during recent years necessitated the prohibition of certain classes of engine from passing over the bridge. This restriction increased the difficulty of working the goods-traffic, and after it had been in effect for some time, it was found necessary to proceed with the strengthening of the bridge. Several schemes were devised for carrying out the work, and in all of these it had to be borne in mind that the traffic, comprising about thirty-five trains per day in each direction, must be uninterrupted during the reconstruction.

Before describing the scheme eventually adopted it may be well to refer briefly to three earlier schemes. In the year 1899, when the Author was Divisional Engineer of the North Wales section of the London and North Western Railway, he carried out under the direction of the Chief Engineer, then the late Mr. Francis Stevenson, M. Inst. C.E., the strengthening of the Conway Tubular Bridge on the Chester and Holyhead line. Two sets of cylinders were sunk into the bed of the river, the original span of 400 feet being divided by the cylinders into three spans of 45 feet, 310 feet, and 45 feet.

This work suggested a similar method of strengthening the Arpley bridge, and it was proposed to employ two sets of cylinders, 6 feet in diameter and fixed 120 feet apart from centre to centre, thus leaving a clear waterway of 114 feet for navigation. It was thought, however, that the cylinder-piers might encroach upon certain statutory obligations and rights of the Manchester Ship-Canal Company and of riparian owners, and possibly prove a source of danger to the river-traffic. An objectionable feature of the old bridge was the narrow width between the main girders, which this scheme would not obviate, and it was desirable for the safety of men working on the bridge that some alteration should be made. Moreover, in this case the top flanges of the main girders were in short lengths of cast iron with doubtful joints, and were therefore unfitted to resist the tensile stress which they would have had to carry. Hence it did not appear advisable to adopt this method of strengthening.

A second scheme provided for the erection of auxiliary steel lattice-girders on each side of the existing structure, with transverse girders

resting upon the top flanges at three or four points in the span. From the transverse girders the old girders would be suspended by long rods extending for their full depth and through the bottom flange. Except temporarily, during the building of the new girders, there would have been no interference with the waterway under the bridge, as in the first scheme, and but little interference with the railway-traffic passing over the bridge. The limited width of the bridge between the old girders, however, would have been still further reduced.

In the third scheme it was intended to strengthen the two outside or face-girders by increasing their depth, adding about 6 feet of new web and a new top flange, and then by moving each of the face-girders bodily outwards a distance of 3 feet, give a greater clear width on the bridge. Both lines of rails were also to be slued outwards, by this means relieving the old centre girder of a considerable proportion of its load. This scheme would have involved the driving of a large number of piles in the bed of the river to support the old face-girders during the process of strengthening, that is, while the old cast-iron flanges were being removed, and the new webs and flanges were being added and riveted in place.

An alternative would have been the lifting-off and floating-out of each of the main face-girders to a temporary staging alongside the river, where they could have been strengthened in turn; but this would have necessitated the erection of two stages, one on each side of the bridge, an arrangement involving heavy expense and considerable risk.

It was considered that this scheme would not have been by any means satisfactory on completion. The whole of the rivets connecting the old cast-iron flanges would have had to be knocked or drilled out, and this would probably have damaged the old wrought ironwork to a considerable extent. There would have been some uncertainty as to the rivets in the new connections filling the old holes properly, and as to the action of the strengthened girder when settling down to its work.

The scheme eventually decided upon was to remove the existing outside girders and to substitute for them steel lattice-girders, transferring to these a certain portion of the load from the old centre girder by means of overhead girders. It was also arranged to widen out the bridge to give the clearance required by the minimum construction-gauge, and to provide a new floor of steel as shown in Figs. 5 and 6a, Plate 4.

It may perhaps be fair to raise objection against this scheme on æsthetic grounds, in that the old centre girder was allowed to remain

in the structure. The removal of this girder was considered, but a careful estimate of the cost involved revealed the fact that an increase of 50 per cent. would have resulted, owing to the difficulty of maintaining the traffic over the bridge while the work was in progress, and of leaving the navigation below unobstructed. Moreover, the structure would have been no more satisfactory from the more practical point of view of its capacity to carry the traffic than it is to-day.

ERECTION OF THE NEW BRIDGE.

In a bridge of this size the method of erection, or carrying out of the work on site, is of primary importance, and Figs. 3-6, Plate 4, show the sequence of the operations involved.

The new steel lattice-girders after being erected at the contractor's works for inspection, were taken down and forwarded to the site of the bridge in pieces, the heaviest of which weighed 12 tons. Before their arrival on the site arrangements had been made for the traffic to be worked over the down line as a single line. The portion of the overhead bracing over the former up line was then cut away, and a special track of 10-foot gauge was laid over the bridge, to carry a traveller for the erection of the new girder (Fig. 3). Longitudinal timbers were laid to distribute the load over the existing timber floor, and on these timbers and coincident with the panel-points of the new girder the camber-blocks were placed.

The new girder, which weighed 135 tons, was built nearer the face-girder than the centre girder in order to relieve the latter of as much weight as possible, having regard to the subsequent operations which would temporarily put additional load thereon. The weight of the new girder with the traveller and other erecting-plant produced less bending-moment in the old girders than the traffic over the bridge had previously occasioned.

The trucks containing the pieces forming the bottom boom of the girder were run under the traveller, the leading rope from the lifting-blocks of which, after passing through suitable snatch-blocks, was fastened to the draw-bar of a locomotive, which by moving away from the bridge, raised the length of the boom clear of the trucks. The latter were then pushed away by hand, and the steelwork was lowered into its correct position. In this manner the bottom boom was unloaded and laid down complete in 7 hours.

The remaining pieces of the girder were unloaded by a crane standing clear of the bridge and were placed on small trolleys of

18-inch gauge; the steelwork was then run directly under the traveller, hoisted up by means of the blocks, traversed, and lowered into place. The girder was erected and bolted up complete ready for the riveters in $3\frac{1}{2}$ days, a gang of twelve men being employed.

The new girder, which is of the Linville type, presents no special features except that it was built with a camber of 9 inches, this excessive amount being provided to suit the camber in the existing centre girder, to which it was necessary for the new girder to conform, in order to facilitate the fixing of the new floor. After the girder had been carefully adjusted to this camber, the riveting up of the joints was commenced immediately with pneumatic hammers, which were supplied with air at a pressure of 100 lbs. per square inch from an Ingersoll-Sergeant duplex air-compressor, driven by an electric motor. This plant was installed in a temporary engine-house at the north-west corner of the bridge, current being provided from the Corporation mains. After the riveting was completed the camber-blocks were removed from under the new face-girder, and a deflection of $\frac{3}{8}$ inch was observed. The new girder was then moved over close up to the existing centre girder to facilitate the removal of the old face-girder.

The timbers forming the floor were in one continuous length for the full width of the bridge. They were cut through along the centre-line of the bridge by boring upwards with augers from a stage fixed on a barge, which could be readily moved clear of craft passing under the bridge. The half timbers were allowed to fall into the river whence they were recovered by means of boats. In the meantime the parapet had been removed from the outside of the old face-girder, which was now deprived of all support. As the top flange was only 2 feet wide, it proved to have but little lateral stability, so that the operation of raising and sluing it sideways to bring it into line with the rails forming the old up line, preparatory to removing it, was a somewhat delicate operation.

The question of the best method of removing the old face-girder, weighing 108 tons, had been carefully considered. Any idea of floating it away bodily and placing it on a stage built in the river, to be cut up at leisure, was abandoned owing to the cost involved. Ultimately it was decided to place the west end of the girder on a low four-wheeled trolley, built for the purpose, and running on the ordinary permanent way. On a suitable occasion, when the tide served, two barges, on which a staging had been erected, were floated under the east end of the girder at low tide, the height of the staging being so adjusted that with the rising tide the girder was lifted clear of its bearing (Figs. 4 and 4a, Plate 4). In view of

the limited time available, this operation was assisted by having the end of the girder resting on jacks which were opened out directly the barges were bearing firmly against the underside of the girder.

The girder was then drawn bodily across the river and over the western abutment by means of a locomotive. A trestle had been constructed on the towing-path, on which a second trolley was placed in position ready for the reception of the far end of the girder after the barges with their load had reached the opposite bank, when the ebbing of the tide gradually transferred the weight of the girder on to this trolley and left the barges free.

The flood-tide at this place only lasts an hour, and the current is very strong. There is a period of 25 minutes of slack water, followed by an ebb-tide of $1\frac{1}{2}$ hour. In order to guide the barges and to prevent the possibility of their load coming into contact with the new girder or drifting up-stream, ropes were secured to moorings on the river-bank and led to winches fixed on the barges. It was hoped that the floating across might be accomplished during the period of slack water, but owing to the fact that it was not fully realized how very much stiffer laterally the old girder would be when supported on the barges with about 50 feet overhanging, rather more precautions were taken to secure the girder than the event proved necessary. These arrangements hindered the movement of the girder to some extent, and the crossing occupied 45 minutes. The second girder, however, was floated across in 8 minutes, the experience gained with the first girder suggesting several improvements in details.

The bearings (Figs. 7 and 7a, Plate 4) for the new face-girder were next placed in position, and the latter was moved over into its final position.

The abutments had meanwhile been lengthened 9 feet 6 inches by the aid of coffer-dams in the usual way.

Before the flooring carrying the up line could be erected, it was necessary to place the existing centre girder in its final position; to do this the whole of the remaining portion of the bridge—consisting of the centre girder and old down-line face-girder, with the flooring and parapet—had to be sluiced a distance of 2 feet 7 inches, and then raised 7 inches above the original level. The total weight amounted to 320 tons, and the sluing was accomplished by inserting well-greased rails flatwise between the girders and the abutments, and sliding the whole structure over by means of hydraulic jacks. This operation was carried out on a Sunday between trains, without breaking the permanent way.

While this work was in progress, the old face-girder which blocked

access to the bridge from its west end, where the storage-yard was placed, was cut up and loaded into wagons and disposed of as scrap.

The new floor is in two halves, as shown in Figs. 5, Plate 4, and was fixed by a 3-ton steam-crane working on the ordinary permanent way, which was laid on the floor itself as the work advanced. To enable the crane to work over the bridge, the whole of the floor carrying the up line was at first only bolted in position; that it was a very easy type to erect may be inferred from the fact that this operation took only 2 days. One end of the floor is suspended from the verticals of the new girder in the usual manner, while the other end is housed in a 15-inch channel bolted to the existing centre girder, for which purpose a line of $\frac{3}{4}$ -inch rivets through one of the flange-angles was cut out. In doing this a few rivets only were dealt with at one time; the holes were then rimmed out and tight-fitting $\frac{7}{8}$ -inch bolts were immediately driven in and through a corresponding hole in the channel-bar.

Temporary overhead bracing of 9-inch by 9-inch timber struts, with ties formed of chains and adjusting-screws, was inserted between the new face-girder and the centre girder, after which the whole of the traffic was diverted on a Sunday over the up line and the down line was closed, the erection traveller being moved over from one side to the other between trains.

The same sequence of operations was proceeded with on the down-line side, except that before the floor could be erected on this side it was necessary to fix the overhead girders and bracing, shown in Figs. 6 and 6a, Plate 4, for which purpose the new face-girder was moved out beyond its final position and afterwards drawn back when the overhead work had been placed in position.

So far, the effect of the operations had been to increase the load to be carried by the centre girder, and as this was undesirable, additional support was given to it by means of a trestle fixed on the edge of the towing-path.

There were six overhead girders altogether (Figs. 6), and before the flooring on the down-line side was fixed, a pair of brackets (*x y z* in Figs. 5) was attached to the webs of the centre girder immediately under each of these overhead girders (Figs. 5). The web-plates of these brackets passed up through a slot in the flange-plates of the overhead girders, the web of which they eventually formed part of. The plates projected sufficiently far above the top of the girders to permit of a cross-head being bolted to them, and angle-cleats were also secured to them immediately above the top of the overhead girder. The spaces between these angle-cleats and the two central overhead girders were filled with tight-

fitting hardwood packings before the erection of the down-line floor was commenced, thus ensuring that the centre girder should receive some temporary assistance from the new face-girders.

RELIEF OF CENTRE GIRDER.

After the girderwork had been erected, steps were taken to relieve the centre girder permanently of some of the additional load it would otherwise have to support, by transferring such an amount to the new face-girders as would ensure that under no ordinary conditions of loading should the stress in the bottom flange exceed 5 tons per square inch. For this purpose hydraulic jacks provided with gauges were seated on the overhead girders and underneath the cross-heads bolted to the webs of the brackets, which, as before described, passed up through slots in the flanges of the overhead girders. By means of the jacks a preliminary load of 20 tons was transferred from the centre girder to each of the overhead girders, and three locomotives weighing 239 tons were then run on to each line (or 478 tons on the bridge) and placed in the position producing the maximum bending-moments in the main girders. The load taken off the old centre girder was then increased to 49 tons at each overhead girder, under which conditions the stress in the bottom flange of the centre girder is approximately 5 tons per square inch. The engines remained on the bridge while holes were rapidly drilled through the upper portion of the bracket web-plates, to correspond with holes already provided in the overhead girders. Rivets were then inserted, thus ensuring that the correct amount of load had been transferred from the centre girder to the new face-girders. This heavy load on each line of rails will, of course, rarely, if ever, come on the bridge in practice, so that the stress in the bottom flange of the old centre girder will never exceed the 5 tons per square inch.

The deflection of the new lattice girders under the test-load referred to, before the additional load had been transferred to them from the centre girder, was 0·62 inch and 0·65 inch respectively, and after the load had been transferred this deflection was increased to 1·1 inch, or $\frac{1}{2000}$ of the span, which was in agreement with the calculation previously made.

As sufficient hydraulic jacks were not available at the time for the whole of the six overhead girders, helical springs of 1½-inch by 1½-inch steel which happened to be on hand were in some cases substituted for the jacks, as shown in Figs. 5, Plate 4. These springs were previously calibrated in a testing-machine at the desired load, and were afterwards compressed to the requisite extent on the bridge by

screwing down the nuts on screws passing up through them. The lower ends of these screws were fastened to the web-plates of the brackets, while the springs were supported on cross-heads packed up from the overhead girders.

The cross-heads were subsequently removed, and the webs of the brackets were cut off level with the top of the overhead girders, cover-plates being then riveted on. The riveting up of the counter-braces of the main girder was finally proceeded with, and when the work had been completed the down line was reopened and ordinary double-line working was resumed.

PAINTING.

The whole of the girderwork is painted with two coats of red-lead and a final coat of slate colour, except the underside of the floor which has a final coat of coal-tar, containing a small quantity of impalpable quicklime. The Author mentions this method of protecting the surface of the steelwork from corrosion as being the best he has yet discovered, after a long experience and numerous experiments with various classes of paint.

From Mr. William Marriott's Paper on the strengthening and maintenance of early iron bridges,¹ and from the discussion that followed, it would appear that the consensus of opinion as to the best method of protecting girderwork from corrosion was in favour of red-lead and linseed-oil, and this agrees generally with the Author's experience. In this discussion the late Sir Benjamin Baker stated that he did not know how to deal effectually with the undersides of bridges. The Britannia Tubular Bridge, which, it may be mentioned, is in a perfect state of preservation, has for many years past been coated with tar on the top of red-lead, except the outer faces which are painted with white-lead paint, partly for the sake of appearance and partly to reflect the heat. This coating of tar, none of which is exposed to the direct rays of the sun, has proved so effectual that the original red-lead paint, or at all events, red-lead paint that was put on many years ago and of which the Railway Company have no record, may be seen at any part of the bridge by scraping off a little of the tar; and it may be stated that the whole of the structure, comprising 28 acres of painted surface, is absolutely free from corrosion.

The Conway Tubular Bridge has a clear headway above high water of only 18 feet, and owing to the spray drifting down the river, it was

¹ Minutes of Proceedings Inst. C.E., vol. clxii, p. 213.

difficult to keep the underside of the bridge, and particularly the rivet-heads, free from corrosion. In 1893 the Author had the tar on the underside entirely scraped off and the surface cleaned, after which two coats of thick red-lead paint were applied and a final coat of tar to protect the red-lead paint. This has been entirely satisfactory, and it is only necessary to tar the underside every 2 years, the red-lead remaining in perfect condition.

TESTS OF IRON IN THE OLD BRIDGE.

The girders having been in continuous use for 55 years (1853 to 1908) and subjected at intervals to stresses amounting to 8 tons per square inch in tension, it was a matter of considerable interest to ascertain the limit of elasticity and ultimate strength of the iron in the flange and web-plates, with a view to see whether the metal has deteriorated to any appreciable extent.

Test-pieces were accordingly cut from various sections, and from the results given in the Appendix it will be seen that little or no change has taken place in the strength of the material.

The contractors for the steelwork of the bridge were Messrs. Cochrane and Company of Dudley. The whole of the erection and work at site, with the exception of the riveting up of the steelwork after it had been placed in position, was carried out by the Railway Company under the direction of the Author, acting for the Chief Engineer, who at that time was Mr. E. B. Thornhill, M. Inst. C.E. The Author's engineering assistant, Mr. W. J. Fuller, acted as Resident Engineer.

The Paper is accompanied by eight drawings, from which Plate 4 has been prepared, and by a number of photographs.

APPENDIX.

TESTS OF IRON PLATES, ANGLES, ETC., REMOVED FROM OLD MAIN GIRDERS.

Description.	Dimensions of Test-Piece.				Area.	Elastic Limit.	Ultimate Stress.	Ultimate Elongation on 6 Inches.	Contraction of Area.	Ratio of Elastic Limit to Ultimate Stress.	Appearance of Fracture.	Remarks.
	Length between Centres.	Width.	Thickness.									
			Inches.	Inches.								
Plates, lengthwise, <i>a</i>	6	1·636	1·021	Square Inches. 1·670	Tons per Sq. Inch. 16·8	Tons per Sq. Inch. 23·9	Per Cent. 9·5	Per Cent. 9·0	70	80% Crystall. 20% Fibrous	} Top flange-plate. Bottom flange-plate. Web-plate. Top flange-plate.	
" "	6	1·773	0·822	1·457	13·2	20·6	13·0	13·8	64	Fibrous		
" "	6	1·758	0·403	0·708	16·2	20·5	7·0	12·5	79	"		
" "	6	1·588	1·019	1·618	15·8	22·7	17·0	17·0	70	"		
				Average	15·5	21·9		13·0				
Plate, crosswise, <i>a</i>	6	1·636	1·025	1·677	17·0	17·5	1·0	4·0	97	75% Fibrous 25% Crystall.	} Top flange-plate. Bottom flange-plate. Web-plate. Top flange-plate.	
" "	6	1·770	0·830	1·469	13·6	17·7	5·0	7·0	77	Fibrous		
" "	6	1·758	0·401	0·705	16·3	18·5	3·0	2·5	88	"		
" "	6	1·586	1·020	1·618	15·4	19·1	3·5	3·5	80	"		
				Average	15·5	18·2		4·2				
Main angles . . .	6	1·768	0·673	1·190	18·1	25·2	7·0	5·0	72	Crystalline	} 3½ ins. × 3½ ins. × ¾ inch angle. Bottom flange angle. Top flange angle.	
" "	6	1·761	0·679	1·196	15·1	23·2	13·0	19·0	65	Fibrous		
" "	6	1·760	0·641	1·128	17·7	25·7	13·0	10·5	69	Crystalline		
				Average	16·9	24·7		11·5				

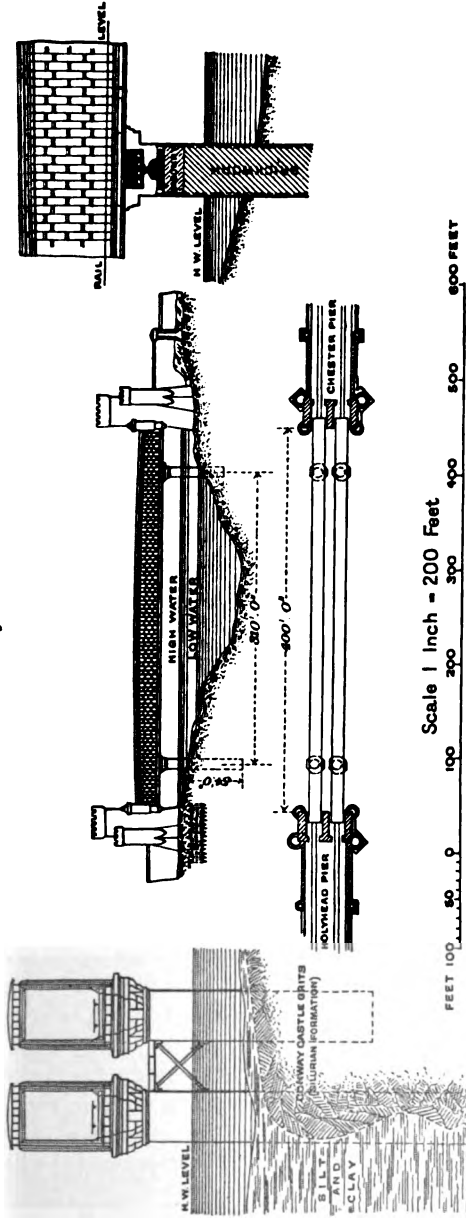
Discussion.

The President. The PRESIDENT moved a vote of thanks to the Author for his two interesting Papers, remarking that it was particularly useful to have Papers of this kind, since, as everyone knew, repairing and strengthening old work was much more difficult than building new work.

The Author. The AUTHOR showed and explained a number of lantern-slides illustrating the work in connection with both New Street station and Arpley bridge. He also exhibited three plaster casts of portions of the original tie-bars of New Street Station roof, and the 2½-inch pin sheared off under test, as described in the Paper. As mention was made, in the second Paper, of the Conway tubular bridge, he also exhibited two slides of that structure. It was the first large tubular bridge ever constructed, the Britannia bridge having been built afterwards. The Britannia tubular bridge had one advantage over the Conway bridge in that it consisted of a long, continuous girder over three piers and two abutments, whereas each tube of the Conway bridge was originally an independent girder resting on two supports only. About 10 or 12 years ago, on taking out the stresses of the Conway bridge afresh, it was found that for modern locomotives, boiler-trolleys, etc., they were somewhat higher than was desirable, and it was accordingly decided to strengthen the bridge by sinking cylinder-piers in the bed of the river, as shown in *Figs. 1*. Borings were taken in the first instance at the centre of each of the proposed piers, and in every case rock was found very near the surface. No difficulty was anticipated in getting down to the required depth, but in sinking the third cylinder it was discovered that the rock had a very curious shape. It went down in a zigzag line until solid rock was found at a depth of 54 feet below the ground-level, the material on one side being soft silt and clay, and the rock consisting of calcareous grit containing copper pyrites. He had placed on the table a piece of rock brought up from a depth of 54 feet when working under an air-pressure of 31 lbs. per square inch. Those who were acquainted with Conway and its neighbourhood would probably have noticed that the span of the tubular bridge was 80 or 100 feet longer than that of Telford's suspension-bridge adjoining, and he had been asked on more than

The Author.

Figs. 1.



STRENGTHENING OF THE CONWAY TUBULAR BRIDGE.

The Author. one occasion why that was the case. The following reason had been given by the late Mr. Edwin Clark:—¹

"The rock on which the tower at the Chester end of the suspension-bridge is built has fortunately face enough to allow of the construction of the abutment for the new bridge, with its river front in the same line. But the rock on the mainland" [i.e. the Holyhead side of the bridge] "shelves off so rapidly beneath the Castle walls, that not only was it requisite to fall 85 feet farther back than the line of the tower of the suspension-bridge to obtain a foundation, but even in this position it was necessary to build a portion of the southern flank on piles driven 15 feet into the beach, to reach the rock beneath. The least span that could thus be obtained was 400 feet. . ."

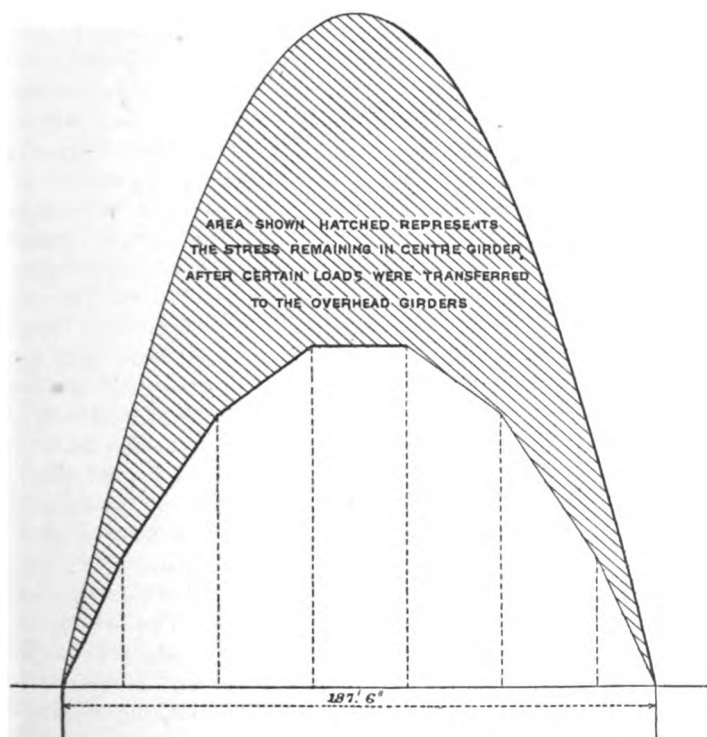
In order to put the weight on the new piers it was necessary to lift the tubes, which weighed more than 1,600 tons each, and he was rather astonished to find that in order to lift the end of the tube quite clear of the abutment it was necessary to lift the tube $2\frac{3}{4}$ inches over the pier; in other words, there was a $2\frac{3}{4}$ -inch bend between the end of the tube and its position over the pier. When the tube was lifted quite clear, the heat of the sun had the effect of bending it 1 inch laterally. It had been noticed previously that the effect of the sun on the tube itself was to lift it vertically 0.81 inch, but there had been no suspicion that the lateral movement would be so considerable. The deflection of the tube by a passing train before the strengthening was 1.49 inch, but when the strengthening had been completed the deflection was reduced to 0.51 inch, or practically one-third of the former amount. With regard to the Arpley bridge, he would like to explain a little more the operation of transferring some of the load from the centre girder to the outside girders. The bridge was loaded with the heaviest load that could be obtained, namely, six of the London and North Western Railway Company's heaviest locomotives, the share of the load taken by each of the three main girders being a simple leverage proportion, and therefore readily ascertained. In order to limit the stress in the centre girder to 5 tons per square inch, a known portion of the load then on the bridge was transferred to the outer girders by lifting the centre girder bodily as described in the Paper. It was impossible to say what the stresses were when the bridge was not fully loaded, as it depended on the relative deflections of the three main girders and the cross girders. It was known that when the bridge was fully loaded the stress in the centre girder could not exceed 5 tons per square inch in tension, and in the outer girders $6\frac{1}{2}$ tons. The change in the stress was perhaps better shown by *Fig. 2*. The parabolic curve represented what the stress

¹ E. Clark, "The Britannia and Conway Tubular Bridges," vol. i, p. 10. London, 1850.

would be if no load had been transferred to the outer girders, and The Author. the shaded part represented the stress remaining in the centre girder after part of the load had been transferred. In order to prevent corrosion at the bearings, which frequently happened in bridges, the rollers worked in a bath of oil, for which luxury the railway-company had to pay a small royalty.

In the station-roof the expansion due to increase of temperature

Fig. 2.



took place vertically rather than horizontally, it having been discovered, in carrying out the strengthening, that the variation in height of a particular principal was more than 1 inch. He desired to explain why a permanent steel bar had been adopted for strengthening the roof in preference to a steel hawser. There were three reasons. In the first place, he considered that a series of wire ropes, thirty-six in number, stretched across such an important roof from one end to the other, would entirely spoil its appearance. That might be said to be only an aesthetic objection ;

The Author. but there were more practical reasons. It had to be assumed, first of all, that there was a possibility of one of the old tie-bars failing, even though relieved to some extent by an auxiliary rope or bar; the new steel bar would take that additional stress without any appreciable extension, while a steel hawser would not. The stress that would be put upon it suddenly would be at least 50 tons, and it might be assumed that if a wire rope had been used an ultimate breaking-stress of not less than 200 tons would have been specified, in order to give a factor of safety of 4. What would the extension of that rope be, under a load of 50 tons applied suddenly? There appeared to be very little information in text-books or in the Proceedings of The Institution on the extension of wire ropes under stress, but there was an excellent Paper on the subject in the Transactions of the American Society of Civil Engineers.¹ Percentages of extension at one-third of the breaking weight were given for three different classes of steel—straight wire, coiled round wire, and patent locked wire. In the case of straight wire the extension at one-third of the breaking-weight for special steel was 0·15 per cent. and for plough-steel 0·22 per cent. In the case of the coiled round wire the extension of special steel was 0·26 per cent. and of plough-steel 0·28 per cent. In the case of patent locked wire the figures were 0·23 per cent. for special steel and 0·27 per cent. for plough-steel. Mr. W. G. Kirkaldy had kindly sent him the elongation at various loads of a rope which eventually failed at 190 tons, and from the information given he found that the extension at one-fourth of the breaking-weight was 0·27 per cent. In the case of the New Street roof that would mean an extension of 7 inches. Messrs. T. and W. Smith, of Newcastle-on-Tyne, had made, for a crane at Woolwich Arsenal, a wire rope having a breaking-weight of 246 tons. The extension at 80 tons on 6 feet was 0·25 inch: at the same ratio, that in New Street Station of 212 feet span would be 8·8 inches. But, taking the extension at one-fourth of the breaking-weight, or 61·5 tons, it would be 6·76 inches, practically agreeing with the results of Mr. Kirkaldy's test. Messrs. W. J. Glover and Co., of St. Helens, who made the rope for the Widnes and Runcorn transporter-bridge, stated that a similar rope with a breaking-weight of 200 tons would have an extension of 0·5 per cent. under a steady load of 50 tons; that would mean about 13 inches under the steady load at New Street. What it would mean under a load applied suddenly

¹ G. S. Morison, "Suspension Bridges—A Study." Trans. Am. Soc. C.E., vol. xxxvi (1896), p. 359.

it was difficult to say. In the event of failure of a tie-rod, the sudden stretch, by at least 7 inches, of a roof-principal one end of which rested on a cast-iron column 30 to 35 feet high, would, in his opinion, be a very serious thing for that roof. He could only say that he would not like to be underneath it at the time. His third reason was connected with maintenance. It was difficult to see how corrosion was to be prevented in the interstices of a wire-rope exposed to such a vitiated atmosphere as that which prevailed underneath a railway-station roof. It was claimed, he believed, that a rope could be encased in lead tubing hermetically sealed, but he was a little sceptical as to the sealing. Iron or steel sash-bars of a roof were sometimes encased in lead, but from his own experience he knew that corrosion went on underneath almost, if not quite, to the same extent as before, with the disadvantage that it was inaccessible to the painter. From the point of view of a maintenance-engineer he could not advocate too strongly the importance of so designing ironwork that, unless corrosion could be absolutely prevented, the metal should be at least accessible to the paint-brush. The Author.

Mr. ERNEST BENEDICT noticed that the tests made of the bridge were static tests, that was to say, no loads were run over it to ascertain the deflection due to moving loads. Certain authorities, to the great joy of bridge-contractors, insisted upon an addition of 30 per cent. for rolling loads or impact, and he would like to know whether the Author had made any allowance for rolling loads beyond what he made for the static loads. Mr. Benedict.

Mr. WILLIAM MARRIOTT agreed with the President's remark that reconstruction was more interesting and difficult than new work. He knew that engineers abroad, in Germany and elsewhere, looked with special interest on such Papers. There were in the Papers many important points worthy of discussion, but he desired to confine himself to two, one of which was in connection with the Table of tests in the Appendix to the Paper on the Arpley bridge. When it was necessary to drill old structures, the drills often wore out quickly, showing that the material was very hard; and that sometimes raised doubts as to the effect that age had had upon the material. The test-results given by the Author would, he thought, go far to set such doubts at rest. The second matter he wished to refer to was the question of protective coatings. The Author's statement as to the Menai bridge and the protective coating of red-lead and coal-tar was exceedingly interesting. Coal-tar was not a very aesthetic substance, but it was very useful, and Mr. Marriott had found it to be the most serviceable coating he could Mr. Marriott.

Mr. Marriott. apply. The Menai bridge was an iron bridge, and personally he wished all bridges were of iron. Probably he was in a minority in taking that view, but when it came to the question of maintenance, he thought it was now firmly established that iron rusted far less than steel. When his Paper on "The Strengthening and Maintenance of Early Iron Bridges" was read about 6 years ago,¹ a very decided opinion was expressed that steel did not rust any more than iron. On one railway there were two gentlemen occupying high positions in the Engineering Department and holding diametrically opposite views, and it scarcely need be said that one was a maintenance engineer and the other a constructional engineer. Now, however, it was fairly well established that steel did rust much more than iron, and no doubt the chemist would say that the cause was segregation. Recently he had turned up an old Paper that had helped him very much to understand the subject. At the Engineering Conference of 1907, in Section IV (Mining and Metallurgy), a Note on "Segregation in Steel" was read by Mr. J. E. Stead, F.R.S. It showed that there was segregation in steel, both of phosphorus and of sulphur—and perhaps also of carbon—but principally of sulphur. Occasionally—not very often, he was glad to say—there was segregation in rails, and a sulphur-print of a rail could be taken, showing the segregation. The end of the rail was polished and burnished, and then had applied to it a piece of ordinary photographic paper dipped in a weak solution of sulphuric acid; the paper was squeegeed on and left for about 5 or 6 minutes, and was then fixed like a photographic print. If segregation of sulphur had taken place, traces of it would be shown on the paper. Of course that could not be done to a bridge; he had tried it and failed, because pitting in steel was so deep that by the time the pit was planed out all the segregation was also removed. In Yorkshire, recently, in company with a friend, the district engineer, he had amused himself by trying to judge by appearances whether certain bridges were of iron or of steel. His companion rather laughed at the idea of its being possible to tell that from the nature of the rust on the outside. After looking carefully at one bridge, Mr. Marriott gave his verdict that it was of iron. His friend was sure it was a steel bridge, as he knew it had been put up by a "steel" man. But when the papers and invoices were turned up in the office, it was found that the bridge was an iron one. Iron rusted in quite a different way from steel. Steel rusted in pits, forming holes, and was

¹ Minutes of Proceedings Inst. C.E., vol. clxii, p. 213.

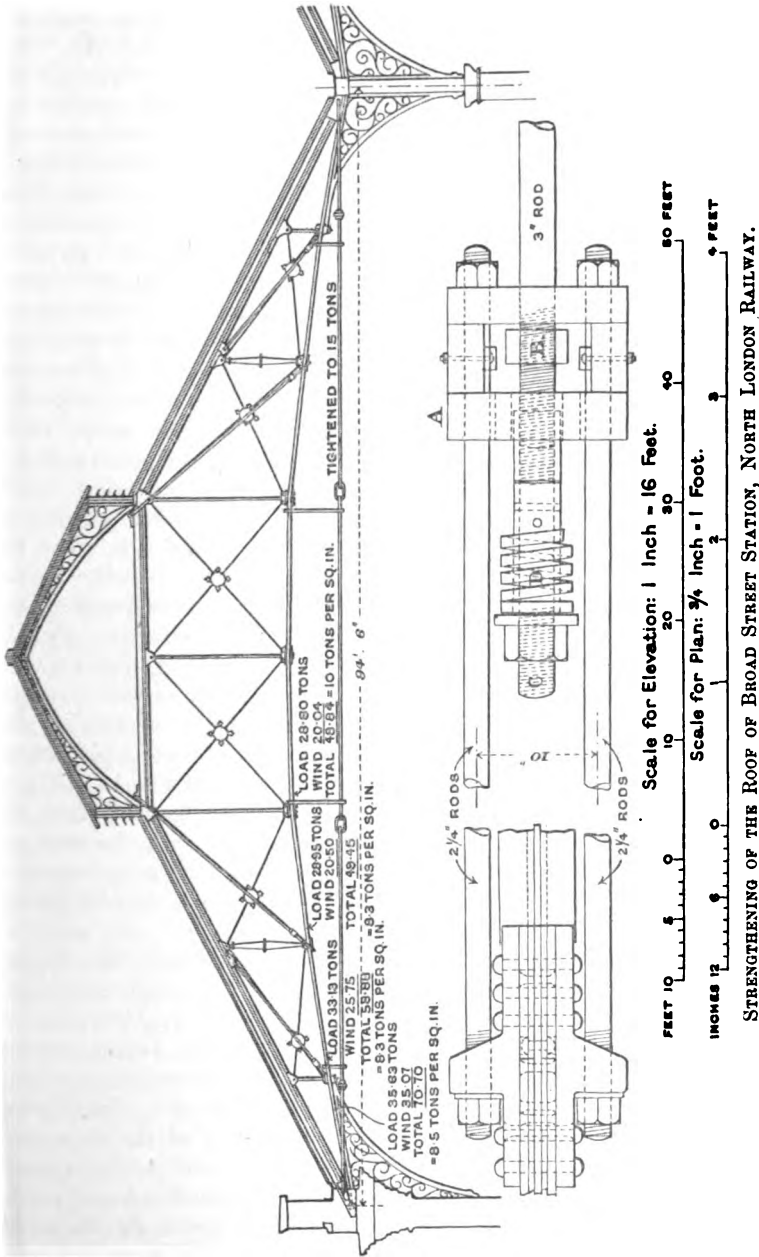
very dangerous. He discovered a remedy some time ago in a scrap Mr. Marriott. of paper which related that a gentleman had had on his table for many years a steel weight, which, although exposed to the atmosphere, had never had a speck of rust upon it, owing to its being coated with paraffin wax. On reading that he tried the method. He burned out all the rust pit-holes with a torch, brushed them well, and then rubbed paraffin wax into them. That was done 7 years ago, and careful examination showed that there had been no recurrence of the rust which was so troublesome in steel. As was well known, steel rusted at the same spot time after time, even throwing the paint off. Another simple coating he had tried was sand-paint. His attention was drawn to a building at Whatstandwell that had not been painted for 28 years, and he found that it had been sand-painted. That was an old method, very good for woodwork. He had used it for about 6 or 7 years on steel, and it was standing remarkably well. The steel was first painted with two coats of red-lead, then with a coat of ordinary paint, and then dried sand was blown over it with a pair of bellows; afterwards another coat of paint was given and sand was blown on again. By that means a kind of stone coating was formed which was wonderfully impervious and was not thrown off by rust. He had tried it on woodwork and on a girder bridge, and a week ago he inspected the bridge, which had been painted in that way 3 or 4 years previously, and found it in remarkably good order, although it was in an unfavourable situation, being over a salt river and near some chemical works. Sand-painting was, of course, more costly, as more paint was used, but it formed an excellent coating which did not wear off. One important point was to use pure paint.

Mr. A. WOOD-HILL thought it would be difficult to suggest any Mr. Wood-Hill. improvement in the methods adopted for dealing with the works described by the Author. With regard to the Arpley bridge, he had been much struck by the short time occupied in several of the operations, such as bolting up the new outside girders and putting in the steel flooring, and particularly with that fascinating operation in which the second old face-girder took farewell of the river in 8 minutes. The method had been so successful as to suggest that by reversing the steps it might have been adopted for drawing across the new face-girders, by attaching a wire-rope from the locomotive to the river end of the girders. That method would have avoided occupying one of the lines while the new face-girder was being constructed on the floor of the old bridge. He could not help thinking that if the Author had realized how well the method would turn out he would have given it serious consideration for

Mr. Wood-Hill. dealing with the new face-girders. With regard to the Birmingham roof, the Author stated that the ends of the new tie-bars were drawn in to the extent of $\frac{3}{8}$ inch, and he would like to know how much of that $\frac{3}{8}$ inch was attributable to the elongation of the new tie-bar, and how much to the inward movement of the foot of the principal. As far as he could ascertain, from reference to Tables giving the results of experiments upon mild steel within the elastic limit, the elongation in the case of the Birmingham tie-rod under a stress of 24 tons would have been about $\frac{1}{4}$ inch, and perhaps the Author could state whether that accorded with the facts. He had the advantage of seeing the Birmingham work just about the time when consideration was being given to the strengthening of the roof of Broad Street station on the North London Railway, and he borrowed the very ingenious idea of the spring for tightening up the tie-rod. As the Broad Street work (*Figs. 3*) was very similar in character to the Birmingham work, a few observations with regard to it might be of interest. He had adopted round rods instead of flat bars for strengthening the roof. The roof was much smaller than that at New Street station, and it had appeared to him that round rods would detract less from the appearance. Also, he had considered that the fastenings could be designed fairly easily, and that the union-screws in the middle of the span would form a ready means of tightening up the rods. The roof was constructed in 1865 by Messrs. Handyside and Co., and in design it took somewhat the form of a queen-truss. There were two spans, each of 94 feet 6 inches. The stresses in the old tie-rods amounted to a maximum of 5.9 tons per square inch for the dead load only, and ranged from 8.5 to 10 tons per square inch under a horizontal wind-pressure of 40 lbs. per square foot, equal to a normal pressure of 24 lbs. per square foot. The diagonal bracing was all of 1-inch rod, which was considerably too weak in the case of the rods supporting the struts under the rafters. It was therefore decided to strengthen those with two additional $1\frac{1}{4}$ -inch rods, one on each side of the principal, and also to provide a new 3-inch tie-rod. The tie-rod was joined by the union-screws in the middle, and connected at the ends to two bars $2\frac{1}{4}$ inches in diameter, which passed one on each side of the framing to the foot of the principal, and were secured to the foot of the principal by heavy steel cheek-plates, with a projecting lug, riveted through the rafters. The rods were tightened to 15 tons, and a single spring was used, which had been tested to 20 tons. There was some little difficulty in obtaining a spring capable of withstanding that pressure that would fit in the narrow limits available. The first spring did not meet the test,

Mr. Wood-
Hill.

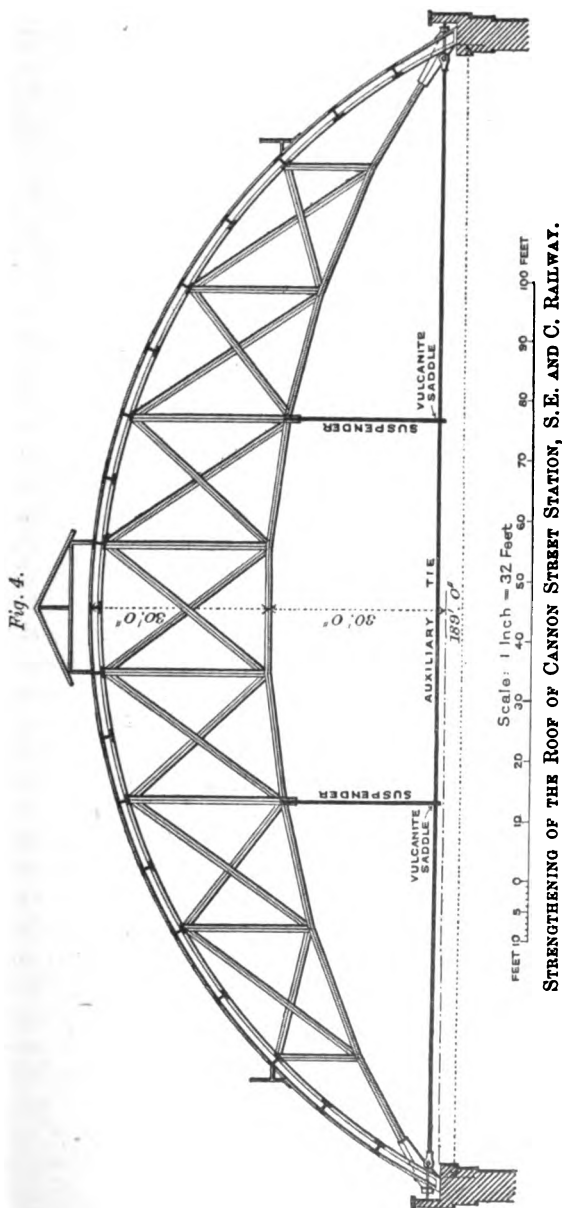
Fig. 8.



Mr. Wood-Hill. but the second was quite satisfactory. A temporary extension-piece, C, was fitted to the end of the 3-inch rod, and the spring D was compressed at one end against a temporary bridging-piece, marked A, and at the other end against a washer and nut. When sufficient compression on the spring—15 tons—had been obtained, by means of the union-screws, the inner nut, B, a permanent nut, was screwed up tightly against the connecting plate, and the spring tackle was taken off. There was no provision for expansion or contraction in the roof, the rafters being bolted to cast-iron shoes, which were secured to the top of the wall by wrought-iron dowels. Some of the brickwork was cut away from the foot of the rafters, and the fastenings of the old tie-rod were examined, but the principals were not lifted as described by the Author. Consequently, the extent to which the stresses were relieved in the old members depended upon any elasticity there might be in the side walls, which were 38 feet high above the rails. Lifting the principals and freeing the intermediate rafters would have necessitated taking out the eaves-gutter, cutting away a considerable portion of the parapet-wall, and freeing the cast-iron brackets under the principals. All that would have added so considerably to the expense that he hardly thought it would have been justified, the main object of the work being to provide a safety-tie in case anything happened to the old rods. A travelling platform was used for executing the work. On the platform were erected wide timber stairs, inclined to the underside of the roof, so that the men could reach the rafters. The stairs were pivoted at their lower end so that they could be lowered to lie horizontally on to this platform when it was required to move forward from one principal to another, and thus the whole gantry with the staircase could pass underneath the principal. A pneumatic plant was used for riveting and drilling. The cost of the work was £2,292, equivalent to £88 per principal.

Mr. Tempest. Mr. P. C. TEMPEST congratulated the Author upon the Papers, which were both interesting and instructive. He had been particularly interested in the description of the strengthening work carried out to the roof of New Street station, Birmingham, because he had had under his own care two roofs of similar construction, namely, those of Charing Cross and Cannon Street stations. The Charing Cross roof had been reconstructed as a low roof of the ridge-and-furrow type, but the Cannon Street roof remained, and perhaps a short description of how that had been strengthened would not be out of place in the discussion. The principal, which was shown in *Fig. 4*, had a clear span of 189 feet, and was much stronger

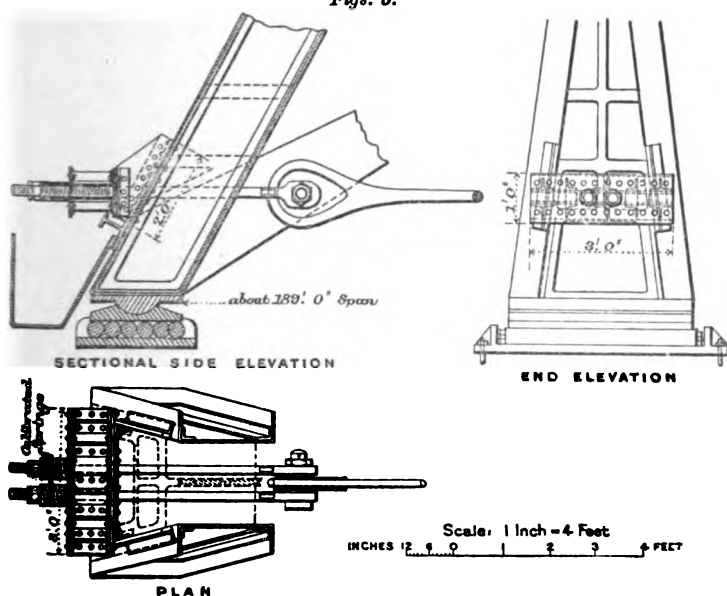
in all its parts than the one at Charing Cross had been. It was Mr. Tempest, carried on walls 6 feet 6 inches in thickness, as against 3 feet 9 inches in the case of Charing Cross. By calculation, all the members were found to be very reasonably stressed under the maximum load. The tie-rod itself, taking dead load, painters' scaffolding, and 40 lbs. per square foot of horizontal wind-pressure, was stressed only to 4.62 tons per square inch; but in view of the experience gained at Charing Cross by the failure of a weld, and the fact that there were many welds in the tie-rods at Cannon Street, it was evident that even that stress would have to be relieved. After due considera-



Mr. Tempest. tion, he came to the conclusion that the best way of treating the roof would be by adding to each principal a horizontal tie, and putting upon it an initial stress to relieve the old tie-rod, making it at the same time of sufficient strength to take, with a proper margin of safety, the whole load, in case of failure of one of the old rods. The tie was a 6-inch steel-wire rope of special construction, composed of seven strands, each of thirty-seven wires. The ropes were made by Messrs. Bullivant and were tested for breaking-strength and elongation under different loads. The breaking-strength came out at 180 tons, and the worst case of elongation, under a load of 50 tons, was 0.19 per cent. There was one rope to each principal, and before erection the ropes were covered with a casing of lead $\frac{1}{8}$ inch thick; after erection, and after the stress had been put upon them, the casings were tested for airtightness by pumps and gauges, and in no case was any leak discovered. The ropes at their eye ends were fixed to screwed rods, which in turn went through the feet of the principals and through stops riveted on the backs of the principals, calibrated springs being inserted between the nuts of the rods and the stops, as shown in *Figs. 5*. The weight of each rope, including its lead covering and the light ropes that suspended it, was 27 cwt. The erection was commenced in July, 1906, one rope being put up each night until completion. A stress of 15 tons, accurately gauged by the calibrated springs, was put on the auxiliary ties, and subsequent examinations had shown that the ties had not extended since they took the original stress, and the diagonal bracing of the principals had not buckled. The effect of putting a stress of 15 tons on the auxiliary ties had been to relieve the old tie-rods of 38.5 per cent. of their stress under normal conditions; and in the event of failure of one of the old rods the maximum additional stress to be taken by the auxiliary ties would be 34 tons. That, according to the results obtained from tests, would stretch the rope $2\frac{3}{4}$ inches. But in view of the fact that one end of each principal was resting on rollers, which, if they did not roll, would at least allow the principal to slide, that was, he thought, immaterial and might be disregarded. The cost of the work had been £3,144, equivalent to about £162 per principal, or £2 11s. 6d. per square of the superficial area covered. For the information of members he had brought the fractured weld of the Charing Cross tie-rod, from which it would be seen that the centre portion of that rod had never been in metallic union, whilst the outer portion had. When pulling down the old principals at Charing Cross the tie-rods were examined very carefully to see if the welds could be discovered, but neither by the naked eye nor

with a magnifying glass were any of them discernible; and it was not until some of the rods were sent to Messrs. Kirkaldy's works, turned down in a lathe, treated with acid, and cut through, that the welds were located. The Author had mentioned that he rejected the idea of using steel-wire rods for the auxiliary ties at Birmingham, because he might get a suddenly-applied stress of 50 tons, which would give an elongation of 7 inches. Mr. Tempest did not quite follow those figures. The total horizontal thrust of the Birmingham station-roof was 50 tons, but it was fair to assume that if the Author had used a steel-wire rope as an auxiliary tie, he would have put an

Figs. 5.



CANNON STREET STATION, S.E. AND C. RAILWAY.

initial stress upon it, as he had done in the case of his flat-bar tie, namely, 24 tons. Had he done the same with wire ropes, that would have left only a possible further load of 26 tons, or about half the load which he had mentioned. That alone would reduce the 7 inches to $3\frac{1}{2}$ inches; and taking the results of the tests in connection with the Cannon-Street roof, and using ropes of the same strength, the $3\frac{1}{2}$ inches would be further reduced to $2\frac{7}{16}$ inches: so that it appeared as though there must be a clerical error in the Author's figures. Another reason given by the Author for not using wire ropes was that he thought steel ropes would corrode even under a

Mr. Tempest. lead covering, and he cited instances, with which all were familiar, of patent-glazing bars that had corroded. Mr. Tempest's experience showed that corrosion of patent-glazing bars—iron or steel bars covered with lead—was due to the outer casing having been pierced by some screw or other fastening which had been used, or to the top of the bars near the ridge not having been sealed properly. After the tests of the casings used at Cannon Street, which were all found to be perfectly air-tight, he thought corrosion was impossible: certainly he had no uneasiness about it himself. At Birmingham one end of the roof-principal was carried on a cast-iron column, and in view of that fact he thought the Author had wisely chosen the best form of tie which he could adopt, because under a sudden load it would not extend quite so much as a wire rope. He would like, however, to have seen the tie fixed horizontally rather than raised, as in Fig. 1, Plate 3, and also to have seen less stress put upon it. He believed that if that had been done the diagonal bracing would not have buckled. At Cannon Street there was a horizontal tie upon which less stress had been put, and the bracing had not buckled.

Mr. Lowcock. Mr. SIDNEY R. LOWCOCK remarked that as he was carrying out some work in a very exposed position and was anxious to protect it as well as possible, he would like to know the proportion of lime put into the tar. The Author mentioned coal-tar containing a small quantity of impalpable quicklime, and the question was, how many ounces to the gallon, or what percentage by weight, was meant by "a small quantity"? [The AUTHOR said about 2 or 3 ounces per gallon.] He wished to congratulate the Author upon the very satisfactory manner in which the work at Birmingham had been carried out so far as the passengers' comfort was concerned. He had had occasion during the whole of the time the work was going on to use New Street station at least twice every week, and beyond the fact that tarpaulins were overhead, and the riveting- and drilling-machines could be heard at work, no inconvenience was caused to anyone, and as far as he could see there was no interference with the traffic.

Mr. Ross. Mr. ALEXANDER ROSS agreed as to the difficulty of reconstruction-work of a heavy character which had to be carried out without interference with a heavy express and other traffic. With regard to the Birmingham roof, it was inevitable, after what had occurred at Charing Cross, that engineers should not place entire confidence in one member for a tie-rod; and he thought engineers and the companies concerned had risen to their responsibility in a very praiseworthy manner. With regard to the Conway tubular bridge, he thought the method the Author had adopted was the most reason-

able and simple that could have been adopted, and one which did not cause changes of stress. He remembered an incident being related to him in connection with the Britannia bridge, which was practically of the same construction. At the time of the Government inspection the leading engineers of the day went down the night before to accompany the Government inspector, and the night turned out to be a clear, frosty one, such as often occurred in North Wales. Early in the morning the bridge was loaded with everything that could possibly be put on. The morning opened bright and clear and the sun came out, and after loading the bridge with several hundred tons, the party went to lunch. On their return they found that the bridge, instead of sagging, as anticipated, had gone up about 2 or 3 inches. The sun striking on the top, and the bottom being in shade, the top had expanded and the girder had assumed a bow shape, the bottom rising. He knew the Arpley bridge well, having had it under his supervision at one time. He believed there was a public street passing on the level at one end, so that the difficulty of getting a free hand to deal with the cross girders was very great. These girders were originally timbers suspended from the bottom flanges. He suspected that the adoption of a simple shallow construction was due to the want of headway above the river. The Author naturally disliked it and wanted it altered, but really there was not so much disadvantage in that construction as appeared at first sight. In the course of his work he had had control over a small railway in North Wales carrying light loads. The line was taken over by a large company with heavy rolling stock, and he was much troubled because, as the old company had no great resources to fall back upon, he was afraid the bridges would have to be reconstructed at heavy expense. These bridges were constructed with continuous cross timbers, 8 inches deep, suspended from the bottom flanges of the girders. To his great relief, they were found to be amply strong enough for the heaviest stock. He took out the old timbers and replaced them with others of the same thickness, placing a tee-bar underneath and suspending the timbers by steel bolts through the tee-bars—thus spreading the weight—and placing a longitudinal timber on the top. That was done at small cost. The bridges were in good condition to-day, and he hoped they would be so for many years to come. To the engineer expediency was everything. It was easy to say that bridges must be reconstructed, but that involved expenditure of capital and loss of dividends; whereas the requirements could often be met at a low cost by some simple method such as he had described. The question arose whether the Author, in adopting a similar structure in the same

Mr. Ross. position as the old bridge at Arpley, had done the most prudent thing. Mr. Ross did not say he had not; it was a very difficult matter to decide. It seemed to him, however, that in course of time—probably not a long time—the line would have to be doubled, and that therefore it might have been wiser to extend the piers, placing a double-line bridge on one side, turning the traffic on to it, and then restoring the old bridge. He could not say whether that would have been the better plan or not, but he mentioned the possibility so that the Author might have an opportunity of explaining it away. He had listened with great interest to Mr. Marriott's remarks on corrosion. He had worked with Mr. Marriott and followed his experiments very closely, and he hoped that gentleman would continue to carry on the tests and would bring them before The Institution as opportunity occurred.

Mr. Thorpe. Mr. W. H. THORPE remarked that he had had something to do with the strengthening of old railway-bridges, and, though he had not used the method adopted by the Author for measuring directly the loads imposed upon new work, there could be no question that, where it was possible, direct measurement was the correct method to adopt. He had had occasion, under the late Mr. J. A. McDonald, M. Inst. C.E., of the Midland Railway, to design and superintend the introduction of centre girders in bridges; that was to say, introducing a centre girder and hanging up the old cross girders to the new girder, in bridges having two outside main girders and cross girders carrying two lines of way. The matter was one of calculation and careful adjustment. The new centre girder itself might be regarded as a spring, because where that was done the load imposed upon the new girder was measured by its deflection when the screwing-up process was carried out. But there was not much assurance in such a case that the precise stresses which it was intended the new girder should take were actually imposed upon it; and the result was only an approximation. The direct method was the one which should be adopted where possible. With respect to the Birmingham roof, it was little to be wondered at that the engineers in charge should wish to relieve their minds of anxiety, when it was considered that in that roof there must be about nine hundred tie-rod welds. With regard to the $\frac{1}{4}$ -inch movement noted by the Author in screwing up the new rods—made up, as nearly as he could ascertain, by the stretching of the new bar and the relief of stress in the old—Mr. Thorpe's calculation showed that the whole of that movement was so made up, thus indicating that there was no appreciable slip at the joints. He would like to know whether that drawing together

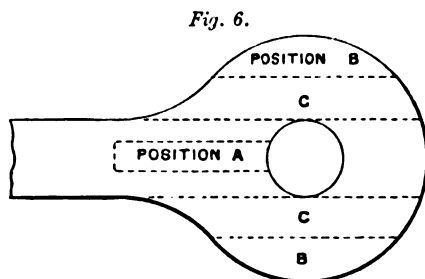
had been $\frac{7}{8}$ inch in every case, or whether it had been sometimes Mr. Thorpe. more and sometimes less. That was a matter of some importance, because it was not always possible to adopt methods of direct measurement, and it was desirable to know whether work executed to a calculated result could be relied upon with tolerable confidence, or whether there was much uncertainty as to the amount of movement and slip at the joints. The rollers under the ends of the principals were said to have been in rather a bad condition, which was not very surprising after so long a period. The first part of the existing roof of Lime Street station (Liverpool) was put up in 1871, and about 1878, during the construction of the second part, he worked under Mr. E. B. Thornhill, then a resident engineer on the London and North Western Railway, and it was his business to be conversant with all that was done in connection with that roof. The older part of the roof had then been up about 7 years, and he was sufficiently curious to see whether there was any movement at the rollers. He found that, for a considerable variation of temperature, the rollers certainly did move very appreciably. He had no means of knowing what the condition was now after an additional 33 years. The utility of rollers appeared to him to be a question of maintenance. No one expected any piece of mechanism to remain in order for 20 or 30 years without any attention whatever, and he really did not see why rollers should be expected to roll if they were entirely neglected. It was a very elementary precaution to house them in and float them up with grease. In the older roofs it was rather a common fault to set rollers under the foot of a principal without any kind of knuckle. That was so at Birmingham. It was exceedingly probable that, with the slight inclination of the rib-foot under stress, the roller nearest the centre of the span had to carry the larger part of the load—or possibly the whole; and it could hardly be expected to work well under such conditions.

Mr. F. HUDLESTON would have liked to hear something more from Mr. Hudleston. railway men as to the difficulty of maintenance. With regard to the stretching of the tie-rods used at New Street, as compared with wire ropes, he did not think tie-rods would be found to stretch very much nowadays, when they were properly drilled through and riveted by pneumatic or other machinery. The permanent set in old riveted work was far greater than it was in modern drilled work, and he thought the permanent set in tie-rods which were solid and drilled should be very small. The method of strengthening the Arpley bridge was very ingenious, but there was one point upon which no speaker had yet touched. If the Author could have managed it, it might have been better to keep the depth of the

Mr. Hudleston. outer girders about the same as that of the centre one. It was well known that the deflection of a girder, other things being the same, varied inversely as the depth. The new side girders were distinctly deeper than the old centre one, and he was disposed to think that under varying loads and varying temperatures the stress taken off the centre girder was rather more a matter of guess-work than would appear at first sight. On a particular occasion, when the Author tightened them up under a definite load and temperature, no doubt the centre girder was relieved of a certain definite portion of its load; but under other conditions the result was more problematical. After all, there was a good deal of uncertainty as to what really happened when old girders were strengthened. In his younger days he had a certain amount of strengthening of old girders to do on the Dock estate in Liverpool, and the method there adopted was similar to that adopted at the Conway bridge. To put a centre prop under an ordinary wrought-iron girder was the cheapest way of doing the work and the most satisfactory, the only thing to do being to arrange it so that it just relieved the girder at the time, and adopt a sufficient factor of safety to cover the uncertainty as to the stresses.

Mr. Kirkaldy. Mr. W. G. KIRKALDY asked for a little more information about the tests of the old materials. An interesting Table was given in the Paper, and personally he would anticipate being in agreement with the Author's conclusion that change had not taken place in the material. Whilst that conclusion was quite in keeping with what he himself had found from experiments made on other old structures, the particular figures in the Table were in some ways rather abnormal. Taking the ratio of the elastic limit to the ultimate strength, there were figures as high as 70 per cent., 79 per cent., and even 97 per cent., and they gave rise to the impression that there had been a change in that material. The normal ratio of elastic limit to ultimate strength for iron was more like 52 per cent.: it might be as high as 55 per cent. or even 60 per cent., but that would be exceptional. He also wished to know if the Author had selected the material for testing from both positions, that was, where it was stressed most and least. He himself always endeavoured to cover the two points, by asking the engineer to follow the members throughout the bridge and select the parts which were least stressed, the results of tests from those parts being taken as indicating the original quality of the iron. Then, by comparing the results with the results of tests of the most severely stressed portions it could be seen whether there had been any change. He had tested recently iron which had been 88 years in service, and it was interesting to know that scarcely any

deterioration had taken place, although the elastic limit had risen. Mr. Kirkaldy. In that case (a suspension-link) it had been difficult to determine the original quality of the iron, but he had managed it by preparing test-specimens from the positions shown in *Fig. 6*, taking the data obtained from position A as showing the original quality of the iron, as indicated in the following Table. Then, by working along the side of the pin-hole, it was found that the metal adjacent to the pin was the most stressed and the elastic limit rose appreciably. Substantially, however, the iron was not changed from its original quality. The link referred to was from Telford's bridge over the Menai Straits. He had also tested iron 30, 40, and 60 years old, and in no case



Position.	Elastic Limit.	Ultimate Strength.	Ratio of Elastic Limit to Ultimate Strength.	Contraction of Area at Fracture.	Extension in 2 Inches.	Appearance of Fracture.
A	Lbs. per Sq. In. 26,900	Lbs. per Sq. In. 50,350	Per Cent. 53·4	Per Cent. 25·2	Per Cent. 21·0	Fibrous.
B	28,000	53,100	52·7	36·0	24·0	" Fibrous laminations or defective welding.
"	22,800	43,260	52·7	20·7	17·0	
C	29,900	52,750	56·7	25·6	22·5	Fibrous.
"	29,400	49,950	58·9	22·9	20·2	Fibrous laminations or defective welding.

had he found any serious change ; so that he agreed with the Author, although the figures in the Paper raised in his mind rather a doubt whether the conclusion held good in the case under discussion. He had been interested in the Author's additional remarks on the question of using a hawser merely as a temporary tie, because he thought the Paper itself was a little weak in not mentioning the Author's reasons therefor. With regard to wire ropes, the makers could supply rope having a small amount of stretch or a flexible rope for crane-work. The extension could be reduced appreciably by laying

Mr. Kirkaldy. the rope differently, but a rope would always be rather a flexible tie. There would always be sufficient additional stretch in a rope when extra duty came upon it to render it undesirable as a tie.

Mr. Wells. Mr. L. FORTESCUE WELLS, referring to the subject of corrosion, asked whether the Author had considered the question of dipping the tie-bars into linseed-oil, and allowing the oil to dry on the bars and form a skin before painting. In the case of Telford's bridge over the Menai Straits, he believed every bar or link had been dipped into a bath of oil heated to a considerable temperature, and had been allowed to stand before being painted; and it was well known that the ironwork in that bridge had lasted wonderfully. Also he wished to know whether there was a good book on protective coatings or paints: he did not know of one. He believed that the chief thing in dealing with ironwork or steelwork was to start well. A great deal of the trouble with bridges originated from the fact that the metal was not thoroughly cleaned, and paint was put on the top of rust. That brought up the question how steelwork ought to be delivered—whether the makers ought to send it out without any paint on or not. He once had some steelwork, consisting of small girders for a bridge, delivered on the ground, and, owing to various difficulties, long delay occurred before the steelwork could be erected. When the girders came to be erected they were in a very bad state. They were thoroughly cleaned, the contractors being paid extra for this work, and as each girder was cleaned it was painted. A few years afterwards he saw the bridge and compared it with other bridges in the same neighbourhood, in connection with which there had been no delay, and therefore the same thorough process of cleaning had not been carried out; and certainly the result was much in favour of the bridge whose girders had been left lying about and then thoroughly cleaned before being painted.

Mr. Marriott. Mr. W. MARRIOTT pointed out that there was an excellent American book by Mr. M. P. Wood.¹ It gave the specifications of all the nostrums that were sold under the guise of paint; and he thought that when an engineer had read that book he would come back to red-lead and a tar compound.

The Author. The AUTHOR, in replying, thanked the President and the members for their reception and appreciation of his Paper. With regard to the deflection of girders under static and moving loads, he could not do better than refer Mr. Benedict to a Paper read before the American Society of Civil Engineers in November, 1898, by Mr. F. E. Turneure, which dealt with several hundred interesting

¹ "Rustless Coatings." New York, 1904.

experiments on bridges, made with scientific instruments to ascertain the deflection of the girders under various conditions of loading.¹ The results were summed up thus:—

"It is quite evident that the diagrams fail to show in any case any general increase in stress or deflection with increase in speed. On the contrary, they show in nearly every case very constant values at all speeds, and, considering the uniformity of the results and the conditions under which the experiments were made, the conclusion seems to be warranted that the effect of speed alone on bridges of spans exceeding 40 ft. in length is of no consequence whatever. For shorter spans the experiments are not sufficiently accurate to be of much value in a discussion of this question."

The Paper was discussed by several eminent engineers in America, and their views coincided with those of Mr. Turneure. He could give several practical instances of deflections of girders under both static and moving loads. He had occasion to test a main-line bridge, the span of which ranged from 57 feet to 63·9 feet, and found that with an engine standing on the bridge there was a deflection of 0·20 inch, and a deflection of 0·22 inch when running over at 30 miles per hour. On a second test being made with the engine running over, the deflection was 0·20 inch, exactly the same as when the engine was standing on the bridge. In another bridge of 39·3 feet span the deflection was 0·24 inch with the engine standing on the bridge and 0·28 inch when running over at 35 miles per hour. On the other hand, a bridge consisting of iron girders with jack-arches and concrete, of 36 feet 2 inches span, deflected 0·05 inch when the engine was standing on the bridge, and but 0·04 inch when the engine was running over. Another bridge of 29·6 feet span deflected 0·16 inch with an engine standing and 0·15 inch with the engine running over. In bridges of short span it was generally found there was an increase in deflection of about 20 per cent. when trains passed over at any considerable speed, but the increased deflection was probably due to one or other, or perhaps to a combination, of the following causes:—

- (a) The effect of unbalanced moving parts of locomotives.
- (b) An imperfectly maintained permanent way (e.g., rails not properly bearing in the chairs, or the chairs on the bridge-timbers, or rails not exactly on the same level, causing slight lurching of the locomotive).

Sometimes rail-joints occurred at or near the centre of the span, and that also was a fruitful source of increased deflection of bridges when loads passed over at any considerable speed. The painting

¹ Trans. Am. Soc. C.E., vol. xli, p. 412.

The Author. question was a big one. A great deal had been said during the last few years on the relative corrosion of wrought iron and steel, and he knew of one or two engineers who constructed their bridges of wrought iron because, they said, owing to the less corrosion of that metal the bridge was less expensive to maintain. He quite admitted that steel did corrode much more rapidly than iron. The evidence of the corrosion of steel rails as compared with iron rails was quite conclusive on that point; but he did not see how it affected the question of maintaining bridges and roofs. The secret of efficient and economic maintenance lay in absolutely preventing corrosion; and once the structure—whether of iron or of steel—had been painted, it was entirely a question of the durability of the paint, and the element of corrosion did not enter into the question at all. He was not aware that paint perished more rapidly on steel than on iron. It was obviously much more economical to prevent corrosion in bridges than to deal with the effect of corrosion afterwards. During the last 20 years he and his predecessor at Crewe, Mr. Harry Footner, M. Inst. C.E., had tested several hundred different paints, and in fact he had forty-eight under test at the present time, to say nothing of a dozen or more tests of different combinations of red-lead, white-lead, and linseed-oil. Some engineers were in favour of raw linseed-oil and others in favour of boiled, while others used both in the same mixture. As a rule, he used raw linseed-oil. Boiled oil was useful because it dried quickly on outside work, but it did not work so freely as raw oil. All experiments with paint in the Maintenance Department of the London and North Western Railway were made at three different places, namely, at Whitmore, where the atmosphere was pure, at Crewe, underneath the station-roof and immediately above the funnels of locomotives, and also in the vitiated atmosphere of Widnes. With regard to the question of the $\frac{1}{8}$ -inch alteration in the length of the tie-rod, in the thirty-six principals the variation ranged between $\frac{3}{4}$ inch and $\frac{1}{2}$ inch, and it was found by actual measurement that the old tie-bars shortened exactly $\frac{1}{4}$ inch. The stress of 50 tons upon a new tie-bar of the Birmingham roof was based on the assumption of complete failure of an old bar, a very unlikely occurrence and one which it was hoped would never take place; nevertheless, the bar was made of sufficient strength to stand the whole stress should one of the old bars break, and the load in that case would be 50 tons as nearly as possible. Even admitting that only a portion of the load was suddenly applied, and that the stretch of a wire rope was $3\frac{1}{2}$ inches, as mentioned by Mr. Tempest, surely this would be calamitous in the

case of a roof of 212 feet span supported at one end on cast-iron columns 30 to 35 feet high! With much pleasure, however, the Author had observed that Mr. Tempest was in agreement with him, inasmuch as he had said that "the Author had wisely chosen the best form of tie which he could adopt." With regard to the "slight buckling" of the cross bracing, the object of mentioning this at all had been to give further proof of the efficiency of the operation of relieving the old rod of some of its load, as the inevitable result of shortening the old bar $\frac{1}{4}$ inch was to shorten each of the thirteen bays of the principal about $\frac{1}{10}$ inch, which length, although almost inappreciable, was satisfactory evidence that the desired result had been obtained. The number of welds in the main tie-rods of the roof was 936. The plan of dipping the bars in hot linseed-oil had not been tried; indeed, it would have been useless in this instance, because after the bars were lifted into position they had to be drilled and riveted, which would have destroyed any oil that might have been applied. In any case he questioned whether linseed-oil would stand long underneath the roof of New Street station. Nineteen of the principals had to be painted every year, four others were painted twice in 3 years, and the remaining twelve once in 3 years.

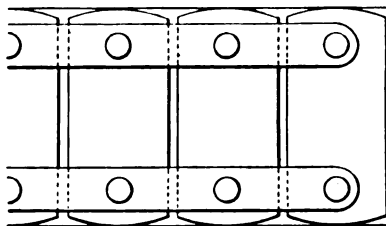
With regard to Mr. Ross's inquiry as to the Arpley bridge, the reason why it had not been widened on either side would be made clear by Fig. 7. At one end of the bridge there was a public street, along which there was an electric tramway, and at the other end were the platforms of Arpley Station; so that in order to widen the bridge on both sides it would have been necessary to reconstruct the platforms and the buildings and roof of the station, and also to take down on the level-crossing side a number of buildings which had probably been built since Mr. Ross knew the bridge. In reply to Mr. Hudleston's question, the new girder was of a more economical depth than the old one, and as the maximum stresses therein were known, the exact amount of the minimum stresses did not appear to be of much moment. As to painting steelwork at the maker's works, he had tried three systems: (1) coating the plates with oil and building up the girders without painting them at all; (2) the usual practice of giving them one coat of red-lead paint; (3) not painting or oiling them, but getting them sent out rusted, more or less with the idea that the rust would help to clear off the old mill-scale. It was difficult to say which of the three systems was the best, but the Company had now gone back to the old system of one coat of paint, which was almost invariably cleaned off after the girders were erected, because

Correspondence.

Mr. DRUITT HALPIN remarked that expansion-rollers were often found to be completely inoperative, owing to their rusting up, as in the case of New Street station roof. This was fre-

quently due to the rollers being of such small diameter that they could cut into the bed - plate without being caused to roll at all. The late Mr. W. Schönheyder had devised a means of overcoming the difficulty which would obviate the disadvantage of the small diameter of the rollers. In this method only segments of a roller

Fig. 8.



were used, and in order to make them act simultaneously they were connected at the top and bottom by parallel bars and bolts (*Fig. 8*). This arrangement, of course, provided means of increasing the diameter of the carrying rollers to any desired size, and thus gave large bearing surfaces, which prevented the cutting-in that necessarily took place with the small-diameter rollers often used.

Mr. G. R. JEBB wished to pay a tribute to the skill displayed by the Author in designing and carrying out the important and difficult works described in the Papers. He thought few realized fully the heavy responsibility which rested on the engineer who had charge of the maintenance of the permanent way, bridges, and other works of a great railway, such as the London and North Western, or the many difficult and complex problems he had to solve from time to time, of which the strengthening of the roof of New Street Station, Birmingham, and the reconstruction of Arpley Bridge were examples. No class of work required more resourcefulness in the engineer than this, where trains were continually passing either over or under the structures dealt with. Of course other methods might have been adopted in the cases under consideration; but it should be remembered that the Author knew all the conditions of the problems better than anyone else, and he must have been impressed by the necessity of running no risks. He had completed the works successfully, at reasonable cost, and without accident; and this result, Mr. Jebb thought, amply justified the means employed.

Mr. Morris. Mr. E. H. MORRIS noticed the Author's unstinted praise of tar as a protective covering for the underside of bridge-floors, but suggested that it must be adopted with judgment. It was true that the undersides of the Britannia and Conway bridges were in an excellent state of preservation, but it must be admitted that these particular bridges presented unique advantages for such treatment. Not only were they constructed of the finest iron procurable, but also their form rendered them absolutely dry, thus putting them upon a footing entirely different from the circumstances of many steel bridges. For instance, the floor of the Menai suspension-bridge, reconstructed in steel 17 years ago, consisted partly of light corrugated troughing and partly of buckled plates carried upon light transverse bearers, which in turn were attached to the suspenders by steel straps passing through slot-holes in the plates. The upper surface was partially, but not continuously, formed in Limmer asphalt; and this want of continuity, together with the presence of the slot-holes, resulted in the floor being anything but dry. The underside was painted with two coats of red-oxide paint and afterwards tarred, the tar being renewed every 4 years. An examination made by him 2 years ago revealed the fact that over a large portion of the underside corrosion was proceeding rapidly, in spite of the tar being fair and intact to all outward appearance. Accordingly he had the whole surface thoroughly cleaned and painted with oil paint. The parts principally affected were the transverse bearers and the sharp convex bends of the troughing, the latter being presumably more readily attacked owing to the molecular disturbance of the metal in the process of bending. In the unexposed situations, such as the soffits of the buckled plates, the original red-oxide paint was sound and intact beneath the tar. A number of samples of the rust-scale had been submitted to chemical analysis which had shown 0·14 per cent. of salt (NaCl), and 0·85 per cent. of sulphuric acid (H_2SO_4) in the rust. The former was undoubtedly due to salt spray, as large quantities of salt were found upon the suspension-chains; but it was not easy to account for the presence of the sulphuric acid. The exposed situation and loftiness of the bridge, the small amount of steamer-traffic in the straits, and the general cleanliness of the atmosphere, would not countenance the suggestion that it was due to smoke; and the only other ready explanation lay in the oxidation of sulphur compounds in the tar. That possibility was certainly discouraging to the use of tar as a preservative in such a situation. Further disadvantages of its use were that, should it become necessary, as in the instance cited, to clean the metal thoroughly, the tar was costly to remove, and it was very difficult to get oil-paint to

adhere properly afterwards; the cost at the Menai bridge of merely Mr. Morris. scraping off the tar was estimated to have been 6d. per square yard.

Mr. J. A. SANER thought that the Paper on the roof at Birmingham Mr. Saner. was particularly interesting, and that the difficulty of carrying out the work without endangering the crowds of people and rolling stock underneath must have caused the Author much anxiety. The only question which appeared not to have been settled so satisfactorily as could have been wished was, how would uneven loading of the roof be met by the two tie-rods and the old cross bracing? The Author stated that some of the cross bracing had buckled, due to the transference of the load to the new tie-rod, and it was evident that there must be also a slight slackening of the old tie-rod. When a high wind or uneven accumulation of snow occurred, the cross bracing, which was buckled, could not come into action until the movement of the arched rib became sufficient to take up the slack; and as this would be cumulative in its action, there would be considerable bending action at the junction of the vertical struts with the old tie-rod. These struts were not pin-jointed, and the arched member would be more severely stressed, particularly at these points, than if the cross bracing had remained taut. He would like to ask whether this additional distortion had been taken into account when computing the stresses in the arch.

With regard to the Paper on the Arpley bridge, Mr. Saner would again like to emphasize the fact that he very fully appreciated the difficulties to be encountered in dealing with a bridge without stopping the traffic over it. It did not, however, seem to be very clearly stated why the new bridge could not have been built entirely enveloping the old one. He suggested that, had the old footpaths been removed and the two new girders—placed somewhat farther apart—been built in cradles outside the old bridge, and the cross girders been put underneath the old planking, the rails could have been transferred to the new bridge one line at a time, with even less inconvenience to traffic than had actually occurred. All of the old bridge could then have been withdrawn, and the new one could have been levelled up to any desired height. In his opinion the stresses in the old girder under the present conditions would be similar to those of a stiffened suspension-bridge. When there was no moving load on the bridge, this girder was being supported entirely by the new girders, and, until the load approached the test-load, was merely so much additional weight on the new girders (provided of course that the cross girders were continuous and able to bear the traffic). Beyond acting as a means of suspension for the centre of the cross girders, the old girder would

Mr. Saner. have nothing to do until the load exceeded $49 \times 7 = 343$ tons. He would be glad to know whether the Author had considered the matter from this point of view.

Mr. Strange. Mr. A. BORROUGHS STRANGE was interested to hear of the Author's success with coal-tar in connection with the painting of the Arpley bridge. This success, however, might be attributed partly to its being used only as a third coating. On the Madras Railway in India, for the sake of economy, tar was used where possible; but experience showed that it lasted a very short time—at any rate, as a protecting agent—on bridges within the influence of sea air; possibly a third coat (two coats only being applied usually) would improve matters appreciably. In March, 1906, he had placed in the verandah of his office, 100 yards from the sea, a steel plate which had been painted in different parts with one, two, and three coats of Torbay paint. The portion with only one coating was now a mass of rust, that with two coats had rust-specks here and there, while that painted with three coats was in perfectly good condition, with the original colours still bright. Bridges in Madras were occasionally prepared for painting by treating them with a kind of Fuller's earth or "soda-soil," deposits of which were found in many places on the line. The soil was mixed with warm water, put on with large brushes, and left for 24 hours. The result was excellent, and he would be glad to know if the Author had tried this method of cleaning metalwork.

The Author. The AUTHOR, in reply, remarked that segments of rollers (or "rockers" as they were technically called) were in common use for expansion-bearings of bridges, and would answer the same purpose on roofs. One drawback to their adoption, however, was their liability to "lock," and he had had two recent instances of this kind which had necessitated the lifting of heavy girders. With regard to the question of using tar on bridges, he would again emphasize the fact that the object of the tar was not to protect the ironwork from corrosion, but to preserve the red-lead paint. Unless the painting with red-lead had been done thoroughly, the outer coating of tar might be worse than useless, as corrosion might set in underneath, with little chance of discovery until the paint either shelled off in flakes or formed blisters. The Conway and Britannia bridges (which he had mentioned specially on account of their being well known) were only two of a large number of bridges on the London and North Western Railway which had been successfully treated in the manner described in the Paper. He had not tried the "soda-soil" found in India, for cleaning metalwork. The scheme suggested by Mr. Saner for strengthening the Arpley bridge, so far as the Author understood it, could not have been adopted on account

of the headway, which was barely sufficient for the traffic on the river and could not be reduced for even a short time. Moreover, the cradles would have involved either pile-driving in the bed of the river or placing the whole weight of the new face-girder on the old face-girder, which was inadmissible. Apart from this, however, the Author ventured to think that the system adopted was demonstrably more economical than the one suggested. The problem resolved itself largely into a question of three main girders with short-span cross girders versus two main girders with long-span cross girders. Theoretically the main girders would be of the same aggregate weight in either case, but as the weights of cross girders varied as the square of the span, the two-girder system suggested by Mr. Saner would be more expensive on account of the heavier cross girders. Moreover, it would be remembered that the centre girder of the three girders in the scheme actually carried out was already there, reducing the cost by about 40 per cent. The additional depth of 18 inches or 2 feet required for the longer cross girders would be quite impracticable at Arpley, apart from cost. With regard to the statement as to the old girder being entirely supported from the new girder when there was no moving load on the bridge, and having nothing to do until the load exceeded 343 tons, the latter figure was incorrect, there being six overhead girders, not seven. Therefore the weight transferred was $49 \times 6 = 294$ tons. The total dead load on the centre girder before any load was transferred was:—

Own weight	150 tons.
New flooring, 186 tons + 2 = ...	93 "
Ballast and permanent way, 120 tons + 2 = ...	60 "
	303 tons.

The amount of the buckling of the cross bracing of the roof of New Street station was so slight that any side pressure from wind or snow would be taken up immediately by the bracing, before any appreciable distortion of the truss could take place.

17 January, 1911.

ALEXANDER SIEMENS, President,
in the Chair.

The discussion on the two Papers by Mr. Wm. Dawson on "The Strengthening of the Roof of New Street Station, Birmingham," and "The Reconstruction and Widening of Arpley Bridge, Warrington," was continued and concluded.

24 January, 1911.

ALEXANDER SIEMENS, President,
in the Chair.

(Paper No. 3910.)

“The Bar Harbours of New South Wales.”

By GERALD HARNETT HALLIGAN.

HOWEVER much engineers may differ on matters of principle or on general details in the design of harbours, they are in agreement upon at least one point, namely, that each case must be treated entirely on its merits. Especially is this so when designing works for the improvement of river-entrances where moving sand is to be dealt with. Permanent or monsoonal ocean-currents, tides, the contour of the shore-line in the vicinity, the winds, the upland water, and the quantity and quality of the stone and timber available, must all be considered; and as these controlling factors cannot be the same for any two places, each harbour or river-entrance must be studied separately. General principles and known laws will, of course, form a guide; but the items to be considered are so varied and correlated that the problem at times becomes very complicated and almost insoluble.

On the coast of New South Wales the problem is simpler than in many other places, but there are still numerous difficulties, and the object of this Paper is to point these out, in the hope that discussion of them may help to solve future problems both there and on works of a similar nature elsewhere.

The Author's 35 years' experience in the service of the New South Wales Government, under which he now holds the position of Hydrographic Officer, has given him an intimate personal knowledge of every harbour, headland, and island on the coast, and the statements in this Paper are based upon that knowledge.

It is proposed first to consider the controlling factors in harbour design in the order named in the first paragraph.

PERMANENT OCEAN-CURRENTS.

The ocean-currents of the Western Pacific in the southern hemisphere are shown in Fig. 1, Plate 5, which has been prepared from the Admiralty current-charts. It will be seen that the current flowing to the north-west, which is caused by the south-east trade-winds, bifurcates at or near New Caledonia: the northern part passes through Torres Strait into the Arafura Sea and thence to the Indian Ocean; the southern branch strikes the Australian coast about Hervey Bay, is deflected southward, and flows along the New South Wales coast as a southward current till it meets the eastward current of the roaring forties coming through Bass Strait. The superior strength and volume of the eastward stream then pushes the southward current to the east till it meets the western shore of New Zealand, along which it flows northward and merges again into the general Pacific current at the most northerly point of that island. On the New South Wales coast the surface current in the offing has a speed of $1\frac{1}{2}$ to 2 knots per hour, and at all the salient points, from Point Danger to Jervis Bay, its speed and direction have been observed.

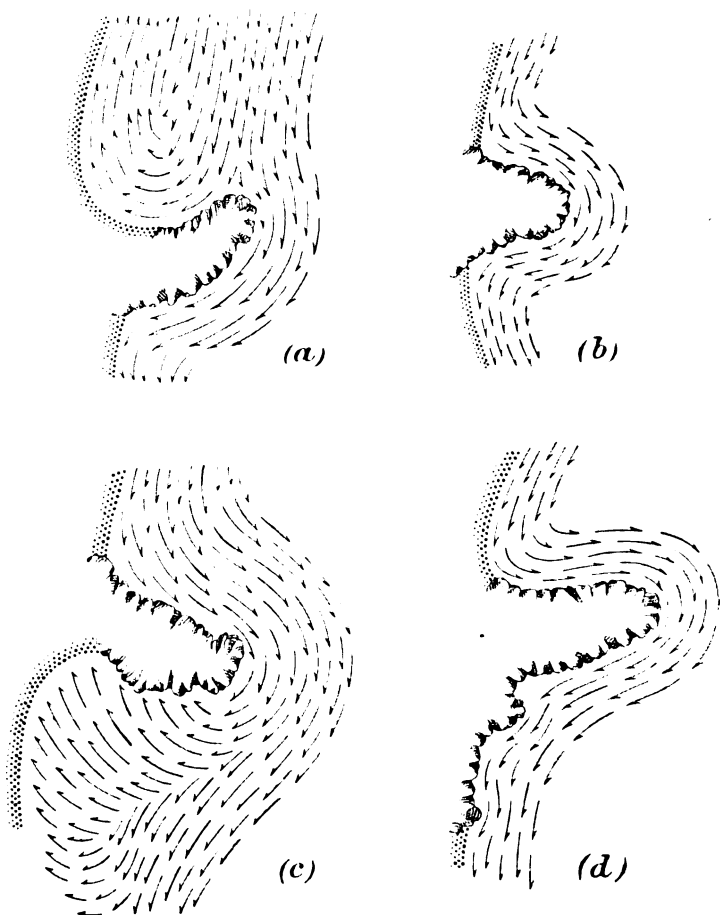
On the littoral the speed is, of course, diminished, and the direction often reversed, on account of the current meeting headlands of varying form. The effect of the contour of the shore-line is shown in *Figs. 2*, which indicate how a current may be deflected or reversed according to the concave or convex form of the headland on its northern or southern side,¹ and on the coast of New South Wales there are several examples of each of these conditions.

In *Fig. 2a* are represented the conditions which control the sand-movement at the Richmond, Nambucca, Macleay and Merimbula Rivers, and at Curl Curl, Manly, and Narrabeen Lagoons near Sydney. In each case the projecting headland or reef to the south causes a reversal of the southward current, which then flows north again along the beach, carrying the sand with it. These counter-currents are necessarily weak and therefore much influenced by the winds, so that it is often difficult to unravel the apparent tangle of results obtained by casual observations at various seasons of the year, unaccompanied by simultaneous records of wind-velocity and barometric readings. Without going into details of all the observations recorded, a study of the topography of the coast will prove that the sand at the places named does travel to the north, for

¹ G. H. Halligan, "Sand Movement on the New South Wales Coast," Proc. Linnean Society, N.S.W., vol. xxxi, p. 623.

the river-mouths are invariably forced against the headland at the northern end of the beach, often flowing parallel to it for a considerable distance ; and this could not occur if the sand travelled from the north. The entrances to Curl Curl and Narrabeen Lagoons, which

Figs. 2.



are occasionally opened to the sea by flood-water, are always closed by sand advancing from the south.

The current-movements when the southern side of a headland is concave are indicated in *Fig. 2c*, such conditions being found at the Tweed and Manning Rivers, at St. George's Head and Disaster

Bay on a large scale, and at Deewhy Lagoon, near Sydney, on a smaller scale. The course of the current when the projecting headland is at right angles to the coast or trends southward is shown in *Figs. 2b* and *2d*. A little more convexity, or a slightly increased length of the headland in the case of the last figure, would result in a counter-current as shown in *Fig. 2a*.

The cause of the counter-currents at the Tweed and Manning Rivers and at Deewhy Lagoon, near Sydney, is that each has a projecting headland to the north with a concave curve to the south, as shown in *Fig. 2c*.

In these cases the return current cannot extend for many miles, as the main southerly stream is constantly endeavouring to maintain its supremacy, and the vortex behind the headland is soon filled in.

In the vicinity of the Manning River the return current extends for about 8 or 10 miles below Crowdy Head, and at this point the stream is very uncertain and sluggish. At Farquhar Inlet to the Manning the sand accumulates from the north or south as the wind prevails and influences the current which carries the sand.

MONSOONAL OCEAN-CURRENTS.

On the coast of New South Wales between Point Danger and Jervis Bay the prevailing wind is from the north-east quadrant, and is monsoonal. In the early summer months—October to December—it blows at 8 to 20 miles per hour; during January and February it is more persistent and frequently attains the force of a moderate gale; and it practically dies away at the end of March or early in April.

An analysis of the wind-records at the Sydney Observatory for the period 1894–1903 is given in the Table on p. 132, from which it will be seen that though the north-easterly winds prevail when all winds for the period are considered, they are in the minority when only gales or strong breezes over 20 miles per hour are recorded.

Practically the same results have been obtained from observations extending over a similar period at the Clarence River.¹ The amount of wind in miles from each point of the compass, i.e. the total travel of the atmosphere (obtained by multiplying the speed of the wind in miles per hour by the time of its duration) in any

¹ G. H. Halligan, "Sand Movement on the New South Wales Coast," Proc. Linnean Society, N.S.W., vol. xxxi, pp. 619–639.

WIND-RECORDS AT SYDNEY IN HOURS: AVERAGE OF YEARS 1894-1903.

	All Winds.				Above 11 Miles per Hour. Gentle Breeze.			
	N.E.	S.E.	S.W.	N.W.	N.E.	S.E.	S.W.	N.W.
Yearly average .	2,588	1,967	2,315	1,860	116·1	108·5	89·6	29·6
Proportion	1·37	1·05	1·24	1·0	3·91	3·66	3·02	1·0
Ratio of easterly to westerly winds .	1·09		1		1·88		1	

	Above 20 Miles per Hour. Strong Breeze.				Above 30 Miles per Hour. Moderate Gale.			
	N.E.	S.E.	S.W.	N.W.	N.E.	S.E.	S.W.	N.W.
Yearly average .	47·4	57·9	59·6	18·4	1·2	13·7	23·5	6·8
Proportion	2·58	3·13	3·24	1·0	1·0	11·5	19·58	5·66
Ratio of easterly to westerly winds .	1·35		1		1		2·03	

direction for each month for the 24-year period 1883-1906, is averaged in the following Table¹:—

TOTAL TRAVEL OF ATMOSPHERE IN MILES.

Month.	S.	S.W.	W.	N.W.	N.	N.E.	E.	S.E.
January .	1,655	433	344	172	296	2,358	1,355	995
February .	1,349	347	263	121	283	2,018	1,112	1,061
March .	1,345	434	422	230	318	1,618	1,022	1,083
April .	1,157	844	1,093	398	354	957	510	467
May .	950	994	1,794	573	312	432	192	359
June .	624	1,008	2,884	982	414	223	186	287
July .	807	1,139	2,937	876	365	191	239	296
August .	999	930	2,351	816	383	506	304	362
September .	873	812	1,815	683	452	974	533	456
October .	1,274	674	1,210	673	463	1,501	900	704
November .	1,516	508	528	316	364	1,999	1,086	922
December .	1,590	354	392	272	333	2,121	1,423	959
Total .	14,139	8,477	16,033	6,112	4,337	14,898	8,862	7,951

¹ H. C. Dannevig, "Coastal Winds and their Influence upon Fish," Proc. Royal Society, N.S.W., vol. xli, 1907.

It will be seen that, with the exception of the westerly winds, which have no great influence on the coast of New South Wales, the north-easterly are the prevailing winds, as regards number, wind-mileage, and duration.

The hardest gales are from the south-east and south-west, and as the wind veers to the east when the anti-cyclone passes, and from that quarter is also directly onshore with unlimited "fetch," it is from there that the worst seas come. The southerly winds rarely last more than 3 days, but cases are on record where gales from that quarter have extended over 7 days, with rough seas all along the coast.

It will be interesting here to note the observed effects of the various winds on the ocean-currents and on the sand-movement on the coast.

The north-east wind, blowing in the same direction as the ocean-current, tends to accelerate it, and also to increase the velocity of the counter-currents. In consequence of this the greatest sand-movement occurs on the beaches between October and March, whether the movement is north or south; and the result is that during that period more sand is conveyed to the river-entrances to form bars and shoals.

The southerly winds are next in importance as regards number, duration, and wind-mileage; but as they blow in the opposite direction to the ocean-current, a large part of their energy has to be expended in retarding, stopping, and reversing the current before the effect becomes a factor in sand-transportation. These winds, however, have an important indirect effect, which will be referred to later, upon the formation of bars. They tend to accelerate the counter-currents where such exist; but this effect is largely overcome by the retardation of the main southward current, which, of course, lessens the speed of the counter-currents; so that the ultimate effect of the southerly winds upon sand-movement is much less than would appear at first sight.

During February, 1908, the Author successfully measured the current-velocity off the entrance to Newcastle harbour in a hard southerly gale which had been blowing almost continuously for 5 days. The wind-velocity at the time was 60 miles per hour, the height of the waves from trough to crest was 27 feet, and the observations were made at the low-water slack tide. The results may be regarded as the maximum disturbance due to southerly winds on this coast, such violent and persistent gales being of rare occurrence.

The surface velocity during calm weather was $1\frac{1}{4}$ knot per

hour to the south, but during this gale the speed was 0·9 knot to the north, and at a depth of 18 feet the speed was 0·3 knot, still to the north. Two days after the gale had abated, the current resumed its normal speed and direction. Up to the present, no periodic observations during a southerly gale have been taken, but such data would be interesting and valuable, as showing the retarding effect of wind upon flowing streams. The difficulties in the way are not insurmountable, but the work is by no means pleasant.

From this it may be concluded that, even during the most violent gales from the south, the current is reversed only during the period of the blow; and as these storms are of rare and uncertain occurrence, their effect upon sand-movement *per se* cannot be large. On the other hand, the northerly winds increase the speed of the current and blow for the longest period, and so must have the largest effect upon the travel of sand on the beaches.

Dr. W. B. Dawson, M. Inst. C.E., Superintendent of the Canadian tidal surveys, makes the following statement:—¹

“It would be quite erroneous to suppose that the wind always causes a drift in its own direction. On the contrary, the set is primarily due to the nature of the current; and if it has any definite direction of its own, owing to the tide or other causes, it takes a strong wind a considerable time to overcome this, even with currents such as these, which do not exceed one knot. For example, on anchoring on June 18th, the wind had been N.E. at 20 miles an hour for about 4 hours; yet the current set strongly N.N.W. or into the windward quarter.”

Further examples of this are given by Dr. Dawson from the experience of fishermen while at anchor.

TIDES.

The range of spring-tides on the New South Wales coast is $5\frac{1}{2}$ feet, and of neaps $3\frac{1}{2}$ feet, and the crest of the tidal wave is approximately parallel to the coast. In consequence there is no appreciable tidal current up or down the coast, so that this may be left entirely out of the question as regards sand-movement on the littoral.

An examination of twenty-one of the more important river- and lake-entrances on the coast shows that in seven cases the outlet is forced to the northern end of the beach by sand-movement, and in

¹ “The Currents on the South-Eastern Coasts of Newfoundland, and the Amount of Indraght into the larger bays on the South Coast,” p. 11. Ottawa, 1904.

fourteen cases it is forced to the south. Travelling from north to south the following conditions occur:—

Tweed River forced to the	North.
Richmond " "	"
Clarence " "	South.
Bellinger " "	"
Nambucca " "	North.
Macleay (old entrance) forced to the	"
" (new " ") "	South.
Hastings forced to the	"
Camden Haven " "	"
Manning " "	North.
Port of Foster " "	South.
Newcastle " "	"
Lake Macquarie " "	"
Tuggerah Lake " "	"
Lake Illawarra " "	"
Shoalhaven " "	"
Moruya " "	"
Tuross " "	North.
Wagonga " "	South.
Bermagui " "	"
Merimbula " "	North.

A study of these facts leads unmistakably to the conclusion that sand-movement on this coast is, below high water, governed entirely by the direction of the littoral current. The heaviest seas are caused by southerly or easterly winds, and the prevailing wind is from the north-east; therefore, as the sand-movement is sometimes to the north and sometimes to the south, it is manifest that neither of these can be the dominant factor. The wind, in so far as it increases or diminishes the speed of the ocean-current, has a greater or less effect on sand-movement, but *per se* it can have no effect whatever. Sir John Coode, in his Paper on the Chesil Bank,¹ says: "... the ultimate movement of shingle is always found to be in the same direction as, and never against, the heaviest seas; ..." At the Chesil Bank, on the north-east side of Lyme Bay on the south coast of England, the ocean-current is purely tidal, and so changes in direction every 6 hours; the storm-wind that happens to be blowing in the same direction as the current raises the sea, which in turn stirs up the shingle, and the current, accelerated in speed by the wind, moves this along the beach.

The prevailing wind at Lyme Bay is also the wind which causes the highest tides and the largest onshore waves, and it blows from

¹ Minutes of Proceedings Inst. C.E., vol. xii, p. 532.

the south-south-west; but the movement of shingle along the Chesil Bank and in the bay generally is not in the direction of the prevailing wind but varies in different places and follows more nearly the set of the flood-current.¹

In New South Wales, as before stated, there is no flood-tide current, but an ocean-stream always in one direction and with but slightly diminished or increased velocity due to the winds; it is this current, in the Author's opinion, that must dominate the movement of sand on the littoral.

The function of the storm-wave is not to move the sand and shingle along the beach, but to tear the beach down and convey the sand seaward to a point where the agitation is less violent, or where the offshore current, created by the onshore wind, is not strong enough to move it farther out.

It is only on rare occasions, and then only for a short time, that the wind is sufficiently violent to cause waves to break obliquely on the beach, and these winds on this coast are as often from the north as from the south. There is therefore nothing in normal wave-action *per se* to cause sand to move laterally on the beach. As the large majority of waves break normally on the shore, their function is to move the sand from a short distance above high water to a short distance below low water, or vice versa; and it is during this movement, when the sand is more or less violently agitated, that the current is enabled to transport it in the direction of its flow.

Sir John Coode's statement as to the ultimate movement of shingle should be qualified by the addition of the words "on a shore where there is no continuous ocean-current in one direction."

The point which the Author wishes to make is that it is the current and the current only which causes lateral movement of the sand on the littoral. Where no continuous current exists, the present wind may cause a current sufficiently strong to move the sand, which has been stirred up by the waves, in the direction in which that wind is blowing. The sand may then be said to move in the same direction as the heaviest seas or in the direction indicated by the wind-vane; but it is not correct to say that it is the heavy sea or the wind-vane that causes it so to move. On this coast there are northerly and southerly gales, and onshore and offshore gales, and, as already pointed out, the southerly blows do at times retard, stop, or even reverse the prevailing southward current and thus stop temporarily the travel of sand; but at any one place on the coast the general direction of sand-movement is always

¹ W. H. Wheeler, "Tidal Rivers." London, 1893.

the same. This has been shown so often by measurement and observation as to be beyond all question.

At every salient point the current is to the south, the speed ranging from $1\frac{1}{2}$ to 2 knots per hour between Point Danger and Jervis Bay, and slightly less between the latter place and Gabo Island. Standing on one of these headlands on a bright day, one may often see sand close to the rocks, held in suspension by the water; and by throwing in a piece of sea-weed a distinct southward movement of the water may be detected. The sand in the bays and inlets on the coast must thus work its way gradually around all projections, across river-entrances, and over reefs, until ultimately it meets the great eastward current from the Antarctic Ocean previously referred to. Whence this sand comes and whither it goes need not be discussed here: it is a subject sufficiently important to be treated in a separate Paper.

Mr. L. M. Haupt, in a Paper¹ read in 1890 before the American Society of Civil Engineers, said he believed it would be found that the unceasing action of the breakers upon a receding shore-line, as affected by the direction of the flood-tide and littoral currents, was far more effective than the prevailing winds.

He continued:

"The coast of New Jersey from Sandy Hook to Cape May has been selected for investigation. Here, if the wind-wave theory be correct, and especially if the movement be due to the more violent gales from the north-east, as asserted, the resultant travel of the beaches should be uniformly to the southward; or from whatever quarter the maximum wind force may proceed, the sands should doubtless move to leeward, but always in the same direction for wind from the same quarter. It would seem anomalous, therefore, to find, as a matter of fact, that from a certain point on this coast-line the inlets and beaches are traveling northward, and at other points southward. But when it is noted that these directions correspond with those of the flood tide along this shore, it indicates more than an accidental coincidence."

On the coast-line referred to, there is no definite ocean-current, but, as in many, if not all, similar cases, the current of the flood-tide takes its place.

Many other confirmations of these observations could be quoted if more were needed, but it is sufficient for the present purpose to say that on the coast of New South Wales not one observation can be shown where sand or beach-gravel is moved, except temporarily, in any other direction than that of the littoral current.

It is a matter of common knowledge among pilots and others,

¹ "Littoral Movements on the New Jersey Coast, with remarks on Beach Protection and Jetty Reaction," Trans. Am. Soc. C.E., vol. xxiii, p. 124.

confirmed by records in the Public Works Department, that during the summer months, when the north-east winds are strongest and accelerate the ocean-current, those bars at the entrances to all rivers at the southern ends of beaches carry less water than during the winter season; while the opposite effect is observed at bars at the northern ends of beaches.

SAND IN THE ESTUARIES.

The sand which accumulates in the estuaries on this coast and forms the bars at the entrances must come from one of two sources. It must have been carried down by the floods and freshets—helped, of course, by the ebb-tide—from the upper portions of the river, or it must have come from the sea-coast and have been carried in by the flood-tide.

The rivers on the coast of New South Wales, although in some cases possessing large catchment-areas, are for a considerable portion of their length only arms of the sea. In the rare and uncertain periods when heavy rain occurs, a small quantity of sand is brought down from the thickly wooded hills, and is, of course, deposited in the upper reaches of the salt-water stream. From here to the ocean there is no fall in the river-bed, and the banks are generally 2 to 5 feet above high water. It is clear that no sand, such as is found in the estuary, can be conveyed in a stream having so flat a gradient; even the fine mud which is held in suspension for a considerable time is almost all deposited before it reaches the ocean. Another sign that sand does not travel in any quantity down the river-bed, is that the bottom is invariably covered with a dense growth of sea-weed (mainly *Posidonia sp.*) for several miles from the mouth, and this could not occur if much sand were travelling over it. As a matter of fact, the end of the sea-weed is always regarded as the upper limit of the moving sand, and is a good guide when measuring its rate of advance.

Even a very casual examination of the sand will show that it comes from the beach in the immediate vicinity of the entrance. It is composed of comparatively coarse, clean, rounded grains of quartzite, entirely dissimilar to the alluvium which characterizes the river-bottom, and in the majority of cases it has its origin in rocks of a formation totally different from that of the catchment-area.

TIDAL CURRENTS.

At most of the river-entrances the tidal current has a velocity of 2 to 3 knots per hour, with a range of $5\frac{1}{2}$ feet at spring-tides and of $3\frac{1}{2}$ feet at neaps. As the rainfall is very uncertain, and the discharge takes place at the head of the salt water, which is always a considerable distance from the sea, it is only on rare occasions that the ebb-tide current is appreciably increased, and still more rarely is the flood-tide current entirely obliterated. Under normal circumstances, or, in other words, under those circumstances for which harbour-entrance works must be designed, the quantity of upland water which reaches the ocean, even in the largest coastal rivers, is insignificant, and may safely be neglected when calculating the effect of scour in keeping the entrance clear of sand. That this is so is abundantly clear from current-velocity measurements and from tidal observations at the river-mouths. The velocity of the ebb-stream, except immediately after heavy rain on the catchment-area, is never greater than that of the flood-tide, and the densities of the water at the various stages of the tide are rarely appreciably different. It is thus obvious that no assistance from upland water can be relied upon at ebb-tide, to scour away the sand brought in by the flood-tide.

When the sand, put into suspension by the waves, and slowly moving with the current along the beach, at length reaches an inlet, it is carried into the estuary by the flood-tide. At slack water this sand is deposited within the estuary, and as there are no waves to agitate the sand inside the estuary, the ebb-current, which, as before stated, is only just equal to the flood-stream, is unable to remove the whole of the sand which has been brought in. An accumulation thus takes place, and is the cause of all the trouble to the harbour-engineer.

It is quite clear that, the narrower the entrance, the stronger the flood-tide will be; and the stronger the flood-tide, the more sand will be carried into the estuary. The velocity of the flood-tide can be increased only by contracting the entrance, and decreased only by widening the entrance, and it therefore behoves the harbour-engineer to keep the entrance as wide as the circumstances will permit. If possible, the width should be so determined that the velocity of the ocean-current flowing past shall be greater than the velocity of the flood-tide between the ends of the breakwaters. The sand will then be carried past in the stronger stream and will be harmless. If these conditions can be obtained, all

dredging within the estuary becomes a permanent improvement, and training-walls become necessary only where the channel is to be moved or directed for local reasons.

Where the sand-movement is to the south it will be necessary to build the northern wall out to such a distance as will ensure the current carrying the sand clear of the southern wall, where such is necessary.

Except in the case of the Macleay River new entrance and the Bellinger River, no second wall will be required, the rocky and broken nature of the southern shore being the best means of breaking up the sea that could be devised.

Where the sand moves northward, the converse obviously obtains.

It will, of course, be necessary to design the entrance so as to produce a sufficient degree of tranquillity within the estuary, and as this may involve contraction of the width, it is a matter for decision what amount of sand-dredging the tranquillity is worth. Each set of circumstances must be judged on its merits, the one fundamental principle being kept in mind that, the wider the entrance, the less sand will be taken in by the flood-tide. The scour of the ebb-tide—that will-o'-the-wisp of the harbour-engineer—may be wholly neglected, as under any circumstances it does not come into operation till the harm is done: but if the entrance is made wide enough, it is not wanted, and serves only to worry the mariner.

NATURAL HARBOURS WITH WIDE ENTRANCES.

As no artificial harbours on the New South Wales coast have been carried out on the principles here stated, it becomes necessary to refer to some of the natural harbours in order to demonstrate the truth of them. Fortunately, there are several of these available, and they are shown in Figs. 3, 4 and 5, Plate 5.

The entrance to Sydney Harbour (Fig. 3) is 4,900 feet wide at its narrowest part and has a depth of 90 feet. The velocity of the spring-tide current is about 1 knot per hour and of the ocean-current $1\frac{1}{4}$ to $1\frac{3}{4}$ knot, to the south. The north head projects farther to the east than the south head, so that the tendency of the ocean-current is to convey the sand, held partly or wholly in suspension, past the entrance, and as the velocity of the tide is so much less than that of the littoral current, no sand enters the harbour under normal circumstances, the depth thus remaining unaltered from year to year. During southerly and south-easterly gales, the ocean-current is

temporarily retarded or reversed, and the sand then in suspension may be carried into the harbour and deposited where the current-velocity lessens. This occurs in the eastern channel between Sow and Pigs Reef and Camp Cove, and in middle harbour between Balmoral and Clontarf. The quantity so brought in is not large, for if dredged to a mean depth of 32 feet at low water, the eastern channel, which is about 800 feet wide, shoals to 29 feet in about 10 years, which represents about 12,000 tons of sand per annum. This sand consists of clean, well-rounded grains of quartzite, similar to the beach-sand to the north and south of the entrance; it is mixed with broken shell and undoubtedly comes from the ocean-beach and not from up the harbour.

The entrance to Botany Bay (Fig. 4) is 3,467 feet wide, with a depth of 60 feet, and the maximum velocity of the tidal current is $1\frac{1}{4}$ knot per hour. The velocity of the ocean-current at Cape Banks is $1\frac{1}{4}$ to $1\frac{1}{2}$ knot, so that this entrance may be said to be as near to the ideal width as possible. There is, however, so little margin between the velocities of the tidal and littoral currents that a comparatively moderate gale will reverse the advantage which the littoral current possesses, and sand will be carried in more often, and in larger quantities, than if the entrance were wider. At the entrance to Cook's River, at the north-west corner of the Bay, the sand accumulates slowly, but soundings taken recently do not show any general shoaling of the Bay when compared with those taken by Captain Cook in 1770.

The entrance to Jervis Bay is shown in Fig. 5; it is 11,590 feet wide, with a depth of 120 feet. The tidal current at the entrance is about $\frac{1}{2}$ knot per hour, while the ocean-current at Point Perpendicular is $1\frac{1}{2}$ knot to $1\frac{3}{4}$ knot. As in Botany Bay and Sydney Harbour, the northern headland projects farther seaward than the south head, and all the sand held in suspension by the ocean-current is carried past. That such sand is carried past the entrance to Jervis Bay is clear from a consideration of the following facts. Current-observations by the Author show a movement of the beach sand southward at Crookhaven Head, and again a decided southward movement below Disaster Bay. The sand below Disaster Bay must come from somewhere, and as there is no recent accumulation between Crookhaven Head and Point Perpendicular, it is justifiable to assume that the sand is carried, wholly or partly in suspension, past the Jervis Bay entrance.

Soundings taken by Captain Sidney, R.N., in 1868, and again by the Admiralty in 1894, show no appreciable change of depth in Jervis Bay and no increase or decrease of depth at the entrance.

It must be quite obvious that the ebb-tide scour can have nothing to do with maintaining the great depth at the entrance to this bay. Neither can the sea have any effect in heaping up the sand to form a bar, nor can the upland water, or the absence of it, have any effect one way or the other. Though the bay is wide and the current is so sluggish as to be hardly appreciable, no training-walls are required to deepen channels or fix them, no dredging is necessary, and times of flood or times of drought make no difference whatever in the depth.

In the Author's opinion this can be due only to two causes: first, that headland upon which the ocean-current first impinges projects farther to the east than the other; and secondly, the entrance is wide enough to ensure the flood-tide current being less than the ocean-current, so that no sand can be carried in to cause trouble to the engineer.

There are several other places on the coast where similar conditions exist, but enough has been said to demonstrate the Author's contention that under the conditions which exist on the coast of New South Wales, wide entrances are required and not the narrow entrances which so many engineers have advocated in the past.

The following Table shows the minimum, maximum, and mean depth, the area of watershed, and the area of tidal compartment for each of sixteen of the principal bar harbours of New South Wales during the period June, 1907, to July, 1909. The depths are taken by the pilots every day, if the weather and sea permit, on most of the larger entrances, and twice or three times a week at the smaller or less important places. The returns are sent in to the Public Works Department, where the Table has been compiled.

The absence of any connection between the area of the watershed of the river and the depth of water on the bar at once strikes the observer. The largest catchment-area is that of the Clarence River with 8,505 square miles, and the smallest is that of Crookhaven River, with only 40 square miles; and yet they have the same mean depth of water on the bar, as indicated by 2 years' daily soundings—in spite of the expenditure of £462,000 on the Clarence River entrance and of £9,000 at Crookhaven.

The Richmond River with 2,683 square miles of catchment-area has the same depth on the bar as Wagonga River with but 52 square miles, though the Richmond entrance has had £422,000 expended on it in breakwaters, training-walls and dredging, and Wagonga not one penny. Similar discrepancies will be found on closer examination of the Table, all going to show that the quantity of upland water, even in the largest rivers, is not sufficient to cause any permanent

PARTICULARS OF THE PRINCIPAL BAR HARBOURS OF NEW SOUTH WALES.
June, 1907, to July, 1909.

Name.	Minimum Depth.		Maximum Depth.		Mean Depth.		Area of Water-shed.	Area of Tidal Compartment.
	Bar.	Crossing.	Bar.	Crossing.	Bar.	Crossing.		
	Ft.	Ins.	Ft.	Ins.	Ft.	Ins.	Square Miles.	Acres.
Tweed . . .	4	0	3	8	9	6	418	5,000
Richmond . .	8	0	9	6	11	6	2,683	6,800
Clarence . .	7	9	9	6	13	9	8,500	34,000
Bellinger . .	2	6	2	9	8	6	479	1,640
Nambucca . .	3	3	3	7	8	0	552	2,730
Macleay . . .	6	0	..	9	6	..	4,581	3,750
Hastings . .	6	0	3	0	7	6	1,389	6,400
Camden Haven.	1	7	1	6	4	10	238	7,240
Manning . . .	4	0	5	0	9	0	3,164	6,800
Cape Hawke .	1	3	1	0	6	10	514	
Newcastle . .	21	3	19	6	22	6	8,269	10,280
Lake Macquarie	3	6	3	6	6	0	291	
Crookhaven .	9	0	3	4	13	0	40	
Bateman's Bay.	4	6	..	7	6	..	696	
Moruya . . .	6	2	5	2	8	8	609	
Wagonga . . .	5	0	6	0	12	0	52	1,500

deepening of the entrance. It may, in fact, be wholly eliminated when designing the entrance-works, any possible good that may occur in times of flood being only at rare and uncertain intervals, and the effect only temporary; while, if the entrance is kept wide enough to ensure a low velocity for the flood- and ebb-tide, no congestion need be feared.

In order to ascertain if any definite connection could be traced between the area of the tidal compartment and the width of the entrance, the following Table was prepared:—

Harbour.	Width of Entrance.	Area of Tidal Compartment.	Ratio.
	Feet.	Acres.	
Sydney	4,900	13,775	1 : 2·81
Botany Bay	3,467	14,246	1 : 4·11
Jervis Bay	11,590	28,326	1 : 2·44

It will be seen that at Sydney there is 1 foot width of entrance to every 2·81 acres of water, at Botany Bay 1 foot to 4·11 acres, and at Jervis Bay 1 foot to 2·44 acres. With the exception of Jervis Bay, none of these entrances is so wide as to allow dangerous seas to run far from the entrance, so that, if the Author's contention is correct, when designing future harbours a ratio of 1 foot width of entrance to 3 acres of tidal compartment may safely be allowed ; but unless it is intended to saddle posterity with heavy bills for dredging, it would appear that the ratio should not exceed 1 to 4.

NARROW ARTIFICIAL ENTRANCES.

The areas of the tidal compartments being given in the previous Table, the width required on this assumption for any river on the coast may be calculated.

Thus the Manning River entrance, in the Author's opinion, should be 1,700 feet wide, and Camden Haven 1,810 feet wide, to allow 4 acres of tidal compartment to 1 foot width of entrance. The width in each case is, of course, the distance between the outer ends of the breakwaters, and not necessarily the shortest distance across the entrance.

The plan of the Manning River entrance (Fig. 6, Plate 5) shows both the works proposed in 1885 by the late Sir John Coode, parts of which have been carried out, and the scheme now proposed by the Author. In his Report, Sir John Coode refers in several places to "a true and uniform channel which would be maintained by a scour without the aid of dredging." On p. 9, he says, "I am of opinion that the scour may be relied upon to produce this result without the aid of dredging." From these and other references in the Report, it is clear that it was assumed that the velocity of the ebb-tide would be greater than that of the flood, for otherwise no effective scour in the direction desired could be possible. From information now available it is known that the velocity of the ebb-tide is greater than that of the flood-tide only during the rare, uncertain, and short periods when heavy rain falls on the catchment, and could not have any such effect as was anticipated. The very narrow width of 800 feet proposed by Sir John Coode would cause the velocity of the flood-tide to be much greater than that of the ocean-current, and all the sand which should drift past the entrance would be carried in, and only part of it be taken out again on the ebb. That part which was not taken out would go to form an inner crossing, while the remainder would be deposited at the outer end of the breakwaters, causing the familiar sand-bar.

Further, the design proposed by Sir John Coode does not take cognizance of a continuous ocean-current in one direction that is now known to exist. As before stated, the width of the Manning River entrance, to ensure 1 foot width of entrance to 4 acres of tidal compartment, should be 1,760 feet, but 1,500 feet has been adopted by the Author in order to obtain greater tranquillity within.

The small amount of dredging thus rendered necessary in the future is the price posterity will pay for the additional convenience which smooth water affords.

Having thus far shown the effect of wide entrances of Nature's design in preventing the ingress of sand to estuaries on this coast, it is now necessary to show the effect of contracted entrances, for comparison.

In Fig. 7, Plate 5, is shown the Richmond River entrance, where the breakwaters and training-walls have been carried out on the lines recommended by the late Sir John Coode in 1885. The breakwaters are not yet quite completed, the northern being 339 feet short of the full length recommended, and the southern still wanting 170 feet; but it is not anticipated that any very marked improvement will result when the walls are finished. The net result of the £422,000 expended to date on this entrance has been to fix the position of the bar, which formerly wandered over 5,000 feet of coast; and in so far as this is an improvement, the works have been a success. Something more, however, might have been expected from the expenditure of so large a sum on so small a place.

The area of the tidal compartment is 6,800 acres, as shown in the Table on p. 143, so that the width of entrance, in order to ensure the ocean-current being stronger than the flood-tide, should be at least 1,700 feet. The width recommended by Sir John Coode was 1,000 feet, and as carried out is 1,100 feet; and the result is shown in Fig. 7.

A large bank of sand of marine origin has been formed between the two breakwaters, about 2,000 feet from the entrance, and this bank is continually increasing and encroaching upon the channel, which it has now narrowed to about 200 feet. Through this contracted area the flood-tide rushes at the rate of 2 knots per hour, carrying its load of sand to the inner crossing. The ebb-tide carries part of this sand out again, and deposits it at the entrance in the form of a bar, and as a bank at the end of the northern breakwater. That this sand is fast filling up the estuary may be gathered from the figures supplied by the departmental annual reports.

During the 3-year period, June, 1906, to July, 1909, 252,040 tons
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of sand were removed by dredging from the bar and inner crossing, at a cost of £4,180, but no improvement is reported in the depth at either place at the end of the period; in fact, rather the reverse.

In the report for the year ending the 30th June, 1909, it is stated that, "Of the two remaining bar-dredges, the 'Antleon' and 'Tethys,' the services of the latter have been almost exclusively required to maintain a sufficient depth of water at the important entrance to the Richmond River." As the annual results obtained by the "Tethys" may be stated at 350,000 tons lifted and conveyed to sea, it will be seen that the maintenance of even the present unsatisfactory, if not dangerous, entrance will be a serious charge against the port for all time.

The expenditure, however, may not be thought too heavy if the experience so gained is properly used. It can now be seen that in a river where there is practically no permanent upland water, there cannot be any effectual scour by the ebb-tide. Floods and freshets are so rare and uncertain, and so short in duration, that their effects must be of quite a temporary character, and in the case under notice they certainly have not been sufficient to prevent the steady influx of sand to the estuary, notwithstanding the large catchment-area of the river. Under any circumstances it is certainly not sound practice to allow sand to come in and then design works to scour it out again. It would seem to be more sensible to prevent the sand from coming in at all, and this can be done, in the Author's opinion, only by adopting wide entrances of proper design.

At the Richmond River the present distance between the ends of the breakwaters is 1,170 feet, though the width of the entrance is 1,100 feet. It is still quite possible, of course, to convert this into a wide-entrance port by continuing the southern breakwater, say, 1,300 feet, with a slight curve to the north to assist the northward current. The effective width of the entrance would then be 1,700 feet so far as sand-movement is concerned, but the entrance would present many undesirable features. The most that could be said of such a scheme would be that it was making the best of a bad job.

The works at Newcastle Harbour (Fig. 8, Plate 5) are another example of the erroneous idea that contraction of the entrance causes a scour of the ebb-tide in a river where there is no permanent upland water.

The catchment-area of the Hunter River is 8,269 square miles, and the annual rainfall is about 36 inches, but the distribution of the fall is so unequal that a permanent flow of water is impossible, and it is rare to find any difference between the flood-tide and the ebb-tide velocity a week after a heavy fall of rain.

The area of the tidal compartment is 10,280 acres, so that to ensure 1 foot width of entrance to 4 acres of tidal compartment the width of the entrance would require to be at least 2,570 feet; but the present width between the breakwaters is 1,200 feet, and, as a consequence, enormous quantities of sand are brought in by every flood-tide, which have to be dredged out again at heavy expense. During the year ending the 30th June, 1909, 546,250 tons of sand were dredged on the bar, but no additional depth was reported; the width of the navigable channel was, however, thereby enlarged from 350 feet to 450 feet. The Chief Engineer for Harbours, when giving evidence a few months ago before the Public Works Committee, said: "The bar dredge was away for 80 days last year on other work, and during that time the bar shoaled from $22\frac{1}{2}$ feet to 18 feet, which represents an accretion of about 2,700 tons of sand." On another occasion, the dredge removed about 38,000 tons of sand from the bar in 6 weeks, and the depth was 1 foot more than when it started. It is now proposed to extend the northern breakwater 420 feet "in order to stop the travel of sand into the entrance," but the proposal does not explain what is to become of the sand which is continually travelling south along the Stockton beach. It is admitted that it does not accumulate north of the northern breakwater, and it can only cross the entrance by first entering on the flood-tide and then going out on the ebb; so that it is difficult to see how any extension of the wall can be of use.

In short, it must be admitted that, in the only two cases on this coast where contracted entrances have been tried, the expected results have not been achieved.

At the Richmond and Hunter rivers the entrances are kept navigable only by continuous and costly dredging, and there is no hope of alleviation while the cause remains. The entrance-works were designed on the assumption that the ebb-tide current would be stronger than the flood-tide, and thus cause a scour which would convey to sea the sand brought in by the flood-tide. When two streams of equal velocity, volume, and duration flow alternately and in opposite directions in the same channel, there cannot be any resultant movement in any one direction; but when one of the streams has any advantage over the other, the resultant movement must be in that stream's direction. In the case of bar harbours, the flood-stream has the sand stirred up for it by the waves, and thus carries more in suspension than the ebb-tide, which must depend wholly on its velocity for the sand it can move; and this advantage is so considerable that the velocity of the ebb must be very much greater than that of the flood before it is overcome.

The resultant movement of the sand is thus in the direction of the flood-tide, and it accumulates in the estuary where the speed of the current slackens.

That the current-velocity of the ebb-tide and the flood-tide is equal on the coast of New South Wales has been proved by many hundred observations by the Author on almost every river on the coast.

CONCLUSION.

There yet remains much to be done in the way of observation of the coastal current of New South Wales before one may speak with certainty of its many peculiarities, but sufficient work has already been carried out to indicate its general direction and its influence on shore-formation. As the direction of the sand-movement is practically constant for any one locality, it becomes, in the Author's opinion, the dominant factor in the designing of harbour-entrances. The area of the watershed and the supposed scour of the ebb-tide, due to the upland water, may be eliminated, as may also the existence of a tidal current in the ocean. The number of controlling influences may thus be reduced to three:—

The speed and direction of the coastal current,

The area of the tidal compartment, and

The rise of spring-tides at the proposed entrance.

The problem thus becomes much simpler than on some other coasts where such favourable conditions do not exist.

The Author desires to thank the Hon. C. A. Lee, Minister for Public Works, and also Mr. E. M. de Burgh, Chief Engineer for Harbours and Water Supply, for permission to use the Departmental plans and other information necessary for this Paper.

The Paper is accompanied by eight drawings, from which Plate 5 and the Figures in the text have been prepared.

(*Paper No. 3881.*)

"Sand-Movements at Newcastle Entrance, New South Wales."

By CHRISTOPHER WATKINS KING, Assoc. M. Inst. C.E.

THE works at the entrance to Port Hunter, on the east coast of Australia, have now been completed as far as arranged for in the original scheme, and have been on their trial for the last few years, during which the trade of the port has increased considerably. It would seem opportune at this stage, therefore, to review the effect of the breakwaters already constructed on the sand-movements, with a view to determine the relative advantage of the different walls, and the total effect of the works constructed, taken in conjunction with the dredging-operations. The Author has therefore endeavoured, by a series of contour-plans, to indicate these different results, which are the net outcome of an expenditure, up to the present time, of £277,425.

Port Hunter, or Newcastle Entrance, is at the mouth of the Hunter River, which drains about 8,000 square miles of country. Owing to the projection of Morna Point 20 miles to the north-east, the southward Australian current is of such diminished velocity that its effect on the tidal currents of the entrance is inappreciable. The area of the tidal basin is 6,500 acres, and the tidal waters extend up to 40 miles from the mouth, into the tributary Williams and Paterson rivers. For 10 miles from the mouth the river is divided into two main channels with various subsidiary channels, through deltaic country, which continues to make up during the high floods that carry down large quantities of silt in suspension.

Experiments made by the Author showed that this silt takes 3 to 7 days to settle in perfectly still water, and with any current in the harbour, it requires 4 to 6 months to become solid enough to be dredgable. This circumstance, coupled with the fact

that all sand dredged from the bar is perfectly clean, shows that the silt itself is not accountable in this instance for any shoaling in the entrance-channel, but that, on the contrary, in high flood, when the velocity attained is about 13 feet per second, the silt and large quantities of sand are swept well out to sea and dispersed in deep water.

To the north-east of the old original entrance, as shown on the plan dated 1816 (Fig. 1, Plate 6), there existed an island called "Nobby's," 130 feet high. Between this island and the mainland there was at that time a channel 10 feet deep, which the colliers then used as an entrance in southerly weather. In 1835 a breakwater, built by convict-labour, was commenced across this entrance, to train the current on to the present channel. This wall was completed in 1848. The sailing vessels then met with difficulty in entering during southerly weather, as they lost the wind under Nobby's and at that time steam-tugs were not obtainable. It was therefore decided to blow up this hill, and undermining operations were completed, but, fortunately, wiser counsels prevailed and the explosives were never fired. Nobby's remains to this day, and carries a fine lighthouse 115 feet above sea-level.

Before the closing of this old channel the southerly seas swept round the island, rolling large boulders fallen from the sea-face of Nobby's, which is to-day a perpendicular wall of stratified argillaceous rock. These boulders have been rolled round to the lee of Nobby's, and since the sand has been removed from them during the last 2 years, they are found to be packed closely together. They extend across the channel, gradually increasing in depth from water-level at Nobby's and Stony Point to 30 feet at the north breakwater. These stones, which weigh up to as much as 5 cwt., are now being removed by a clam dredger, and below this layer the Author expects to reach a bed of shale, which will require blasting for its removal.

CHART OF 1816.

This is the earliest plan of the entrance which gives any detailed information about the depths (Fig. 1, Plate 6). It was surveyed before any works for the improvement of the channel had been undertaken, and, in addition to the channel to the north of Nobby's, it shows the channel 10 feet deep between Nobby's and Signal Hill which small sailing vessels frequently used. The dangerous character of the entrance at this date is clearly indicated by the number of wrecks charted on the plan at even this early period of the Colony's history.

CHART OF 1866 (Fig. 2, Plate 6).

This chart shows the 10-foot channel cut off by a breakwater, and the existing line of leading fairway-towers defined. The 21-foot channel was only 150 feet wide off Stony Point. Attention may also be drawn to the sand-spit to the north of Nobby's, running 80 feet north of the fairway-line, and to the two additional wrecks shown on this plan.

CHART OF 1887 (Fig. 3, Plate 6).

In this plan part of the south breakwater is shown as constructed, and also the present inner, or old north, breakwater. It will be observed that the 21-foot channel has improved perceptibly up-stream from opposite Nobby's lighthouse, and the large sand-spit off the Stockton shore is nearly scoured away; this latter action was fully explained by Mr. C. W. Darley, M. Inst. C.E., then Engineer-in-Chief, in evidence before the Public Works Committee in 1895. Opposite the commencement of the south breakwater there is clearly shown the bar, on which at this time there was a depth of 20 feet. It will be observed also that the deep-water channel is tending to follow the south breakwater, and is working over to the south of the fairway-line, which action will be observed to continue till the 1909 plan.

CHART OF 1896 (Fig. 4, Plate 6).

This shows further improvement in the channel, but unfortunately further wrecks on the Oyster Bank and Stockton shore. It may be mentioned, however, that before 1896 very heavy floods occurred in the Hunter River, that of 1893 being the greatest ever known, and these heavy discharges undoubtedly had a good effect in clearing away large quantities of sand. A very ugly shoal, though, is shown as formed to the north-east of the old north breakwater. Another shoal was beginning to form on the fairway-line opposite the end of the south breakwater at that time, where the 8-fathom contour had come in 600 feet, and the depth had decreased from 48 to 24 feet. It was about this time that the Public Works Committee recommended the construction of 3,000 feet of north breakwater and the extension of the south breakwater by 300 feet,

CHART OF 1903 (Fig. 5, Plate 6).

This plan, from the Author's own survey, shows three additional wrecks on the north, and the construction of the breakwater to 2,000 feet out. Two very important sand-spits are clearly defined, which were becoming a source of danger to navigation, both having only 18 feet depth of water at 80 feet from the line. The ebb-and flood-tide used to rush round the end of the north breakwater at the high velocity of 3 knots per hour, forming the channel shown, and deepening the tip-head to 36 feet. The entrance at this time was a dangerous one in heavy weather, the ships entering being liable to be swept over by the seas into shallow ground to the north. A deep channel was still available, however, by keeping close to the south breakwater, outwards from Stony Point.

1904.

In September of this year the barque "Adolphe" was wrecked on top of the old wrecks of "Wendours," "Lindus," "Colonist," and "Cawarra": and an advisory Board of Engineers decided to close the gap between the end of the north breakwater and the "Adolphe," in order to stop the flow of sand from the Stockton Bight, and to keep the ebb-tide current in a straighter course. This was effected by the sinking of the old steam-hulks "Elamang" and "Katoomba," and also of thirteen iron hopper-punts loaded with stone. The positions of these hulks and punts are shown in the next plan.

CHART OF 1905 (Figs. 7 and 8, Plate 6).

During this year the channel caused great anxiety, owing to the perpetual formation of various small shoals at different places. The sand was all alive, but gradually working away, owing to the blocking action of the fan-shaped currents to and from the north; and during this process the shoals kept changing in position and depth, thus requiring constant watching and attention. There was at times as little as 18 feet on the fairway-towers line. In this and the following year good work was done by the sand-pump dredgers "Antleon" and "Tethys" in removing the shoals, but these dredgers were of too small a capacity to effect any permanent improvement.

CHART OF 1907 (Fig. 9, Plate 6).

This survey, taken before the work of the bar-dredger "Jupiter," shows the effect of the north breakwater without any appreciable dredging having been done. The 21-foot channel had now widened out to 450 feet off the end of the north breakwater, and the sand-spit in that portion of the entrance was slowly but surely disappearing. The ebb now ran out in a straight direction, with a velocity of 2·4 knots per hour, carrying the sand seaward; but the flood-tide still came round the end of the north breakwater, carrying large quantities of sand (about 430 tons per day), though with the diminished velocity, at the end of the breakwater, of 2·4 knots per hour instead of more than 3 knots, as in 1904. The south breakwater extension had also the beneficial effect of diverting the force of the heavy seas to the north, instead of in the direction of the "Regent Murray," thus reducing the danger to navigation. It will be observed also that the constant shoaling of the various parts of the channel as in 1905-6 has practically ceased. Unfortunately, however, the outside shoaling, which, in reality, is now the "bar" in the usual acceptance of that term, shows a steady increase below the 4-fathom contour-line.

CHART OF 1909 (Fig. 10, Plate 6).

This plan shows the contours of the entrance in March, 1909, due to the combined action of the north breakwater as far as at present constructed, and the dredging at the bar of 944,050 tons by the sand-pump dredger "Jupiter." The 21-foot contour has widened out opposite Nobby's to 800 feet, as against 200 feet in 1866, and the rocky bottom of the shallow parts of the channel on the fairway-marks line has been completely cleared of sand, leaving the stone bare and allowing marine growth to appear thereon. The removal of a portion of these rocks has now been taken in hand, as already mentioned. The shoal opposite the end of the south breakwater is still in evidence, as indicated by the tongue of the 5-fathom contour coming down to the fairway-line, but during the last 12 months this has deepened from 24½ feet to 29 feet, and is therefore not increasing, though how much of this negative effect is due to the "Jupiter" it is difficult to estimate. This, however, is now the danger-point, causing the seas to break in heavy weather; the motion of the waves alters from oscillation to translation slightly south of the towers line and the seas break right across the remainder

of the channel, with such force that a vessel would have but little chance if caught, being here broadside on for entering the harbour. It is therefore highly essential that this shoal should be removed, and the proposed extension of the north breakwater would certainly assist materially; but it will probably be necessary, in the first instance, to dredge to the maximum depth obtainable, when the breakwater would have a better chance of maintaining the depth (with possibly occasional dredging with a sand-pump), by arresting the flow of sand from the Stockton Bight. The highest waves outside, as measured by the Author, were 16 feet from sea-level to crest, and the corresponding maximum range in the harbour was 7 inches, whereas the range was 11 inches before the construction of the north wall and of the last extension of 300 feet to the south wall, together with the two wave-traps. This substantial decrease is of much importance to vessels lying alongside wharves which are in such close proximity to the ocean.

DREDGING AND BREAKWATER-CONSTRUCTION.

The dredging by the "Jupiter" during the early part of 1908 was principally on the shallowest portions of the channel proper, but the Chief Engineer endeavoured to induce the main channel to move over towards the north to avoid the rock. Dredging was commenced as close up to the north breakwater as safety permitted and extended from the "Regent Murray" seaward for 1,500 feet, or in a north-easterly direction from that wreck. This had the effect of cutting off the supply of sand which continually pours in from Stockton Bight, and the ultimate result of these operations is as shown in Fig. 10, Plate 6, and mentioned previously.

The dredging-plant consists of the self-propelling twin-screw bar-dredger "Jupiter," fitted with twin suction-pumps and trailing pipes, and capable of filling its 750-ton hopper in 20 minutes; the average work per day, dredging and depositing at sea, amounts to 3,000 tons. There is also a small clam dredger engaged in removing the boulders overlying the rock in the channel.

The sinking of the hulks "Elamang" and "Katoomba" and the iron hopper-punts, in January to March, 1905, was an interesting operation. The hulks were originally passenger-steamers trading between Brisbane, Sydney, and Melbourne. The "Katoomba" had a displacement of 1,006 tons, with a length of 250 feet, a beam of 29 feet, and a moulded depth of 19 feet. The displacement of the "Elamang" was 946 tons, length 230 feet, beam 29 feet, and

moulded depth 20 feet. The hulks were first dismantled, trimmed to 9 feet draught, and moored at slack water in the positions shown on the plan; the sea-cocks were then opened and the vessels sank into the sand. The "Elamang" was loaded in position with 340 tons of stone and 1,300 tons of sand. The "Katoomba" was stiffened with 250 tons of stone and strengthened amidships with two encircling chains and struts before fixing, and 1,400 tons of sand were pumped in when she was moored into position. The "Elamang" broke athwartships 3 days after fixing, having settled down 5 feet into sand, and the "Katoomba" also gradually settled into the sand and in 4 months collapsed. The iron hopper-punts were loaded with stone packed into the air-tight compartments and hoppers, and were carefully moored into position at slack water; they were then scuttled one at a time by means of previously-prepared holes in their plates, which allowed the water to enter evenly throughout. It was necessary to moor the punts very carefully, as experience showed that when sinking each of them tried to capsize.

While the hulks and punts were being sunk, the breakwater was pushed forward day and night, and quickly covered the remains of the vessels. These hazardous operations were carried out without mishap of any kind.

The immediate effect of this rapid restriction of access for the tidal currents round the north wall is clearly shown on the contoured plans; the shifting sand was forced seaward at ebb-tide, and a large portion of the sand that came in during the flood-tide was cut off. The ultimate effect was far more pronounced, for instead of building the breakwater into a hole 36 feet deep, which continually advanced in front of the tip-head, the hulks and punts kept the depth at 15 to 20 feet, and enabled the wall to be built more expeditiously and at much less cost.

It has now been proposed by Mr. E. M. de Burgh, M. Inst. C.E., Engineer-in-Chief for Rivers and Harbours, to extend the north breakwater 420 feet, so as to cut off the sand-drift still further, and to extend the south breakwater about 200 feet, so diverting the heavy seas still more towards the north. In this proposal the Author concurs, as all the evidence he has gathered in his study of the matter points to the probability that with this increased length of wall the sand-drift will be kept within easy control, and the tip-head will not be far enough out to catch, and divert into the harbour, the heavy seas which pass the protection of the south wall. This proposed extension of 420 feet of the north wall was first recommended by Mr. C. W. Darley, M.Inst.C.E., Engineer-in-Chief in 1896, under whom the Author had the honour to serve. The ultimate

aim of the Department is to obtain a depth of 30 feet at low water, and with this in view it is anticipated that, the sand-movement question having been dealt with satisfactorily by the proposed breakwater-extension and the assistance of the dredger "Jupiter," the work of removing the rock from 23 feet down to 30 feet will be lightened appreciably.

In conclusion, the Author desires to thank Mr. E. M. de Burgh, M. Inst. C.E., Chief Engineer for Harbours and Water-Supply, for his permission to submit this Paper, and also for his kindly criticism and assistance during its preparation.

The Paper is accompanied by one tracing and a print, from which Plate 6 has been prepared ; there are also two photographs.

(*Paper No. 3897.*)

"Fremantle Harbour-Works, Western Australia."

By CHARLES STUART RUSSELL PALMER, M. Inst. C.E.

WESTERN AUSTRALIA covers about one-third of the island continent, being demarcated politically from the rest of Australia by the 129th meridian of east longitude. On the other three sides it is bounded by the Indian and Southern Oceans. In comparison with the large area of the State, nearly 1,000,000 square miles, the coast-line is very small, being estimated at only 5,200 miles; for although the small portion north of the parallel 18° S. latitude is deeply indented, on the other hand, south of this parallel, the coast is marked by long straight reaches little broken by inlets, the only natural anchorages available, with the exception of Albany and Port Hedland, being roadsteads only partially protected.

THE HARBOURS OF WESTERN AUSTRALIA.

Responsible Government was granted to the Colony in 1889, and the great gold-discoveries of the next few years, with the resulting rapid increase of population, trade, and resources, enabled and compelled the newly enfranchised people to start extensive improvements of their estate, not the least of these being harbour facilities. These works have been prosecuted with considerable vigour, about £2,250,000 having been spent on them in the 10 years or so before 1904, in which year the Author's executive connection with the works ceased.

For the purposes of this Paper, however, attention need be directed to the marine works at Fremantle only, with some slight reference to those at Albany and Bunbury. These ports are the principal trade outlets for the area where the preponderating

settlement has taken place. The other works, distributed along the whole coast, consist of timber jetties, with their accessory tramways and other adjuncts, and have been constructed at places where the small population and trade did not warrant heavier expenditure. These jetties project, in the main, into the open roadsteads mentioned. For this reason, and also because of the great range of the tides and the violent cyclonic storms at many places, each work, though comparatively small, entailed considerable care, in order to make the most of limited means. Nevertheless, such works form a class by themselves; and although the destruction of one of the jetties in a violent cyclonic storm afforded valuable lessons, stress is laid, in the following physical descriptions, only on those features which affected the harbour-works in the south-western portion of the State.

The oldest rocks of the southern portion generally of the State are gneissic or granitoid, assumedly of archæan age, and geologists claim that this group of rocks is more largely developed there than in any other part of the world. They are greatly contorted, and, broadly speaking, now appear at the surface as four parallel belts striking roughly north and south. The third and fourth, or eastern belts, are far inland, except where they outcrop in the extreme south, and although connected so directly with the State's chief source of prosperity, the gold-mines, these belts do not require consideration here.

The second belt, which extends from the Murchison River to the south coast, forms the escarpment at the edge of the great plateau, the Darling range, the western edge of which is situated 20 to 30 miles from the coast. From this range and the inland plateau flow the rivers debouching near and at Albany, Bunbury, and Fremantle.

The first or westward belt also extends from the Murchison River to the south coast. The rocks composing it are comparatively soft, and, whether owing to denudation, attrition, or gradual submergence under the weight of superincumbent and later strata, the rocks of this belt south of the Irwin River are little exposed, except between Capes Naturaliste and Leeuwin, where they form a small range adjoining the coast. At Bunbury a basaltic outcrop and overflow forms an important headland, but the rock is apparently of a much later date than the archæan belt.

The stretch of country west of the second belt, that is, between it and the first belt or the sea-coast, consists of sandy plains, the result partly of the wearing away of the archæan rocks and of the later formations, which have evidently been greatly denuded, and

partly of seashells, including the remains of some of the minutest marine creatures. The deposits forming these sandy plains undoubtedly exhibit the characteristic false bedding of wind-blown material. Generally speaking, they have been more or less cemented below the surface into a sandstone or sandy limestone with irregular patches of purer limestone; but extensive sand-dunes, still exhibiting a tendency to travel, also occur along the western and southern coasts, at the river-mouths, or where the land is low.

Rain falling on this coastal area sinks at once into the ground, and the discharge, even of those rivers which have cut their way from the interior through the Darling ranges, is comparatively small. The many streams, therefore, that issue from these ranges do not now bring down much material, and, indeed, the volume of their waters is so small that the rivers are unable to do more than occasionally clear their mouths of extraneous sand.

In the north of the State the tidal range is large, but south of latitude 22° S. the maximum variation of water-level, and this dependent largely on the winds, rarely exceeds 5 feet, and is often a good deal less.

In the months of January and February strong currents set towards the land in the vicinity of Cape Leeuwin. The general current divides there, one part setting northward and the other part eastward along the south coast at the rate of 1 to 1½ knot per hour. Several days before, and for some time after, the arrival of the north-westerly gales, a strong set to the south is usually experienced on the west coast. Near Fremantle the current generally sets between Rottnest Island and the straggling rocks in a N.N.E. direction during the summer or with a prevalence of strong S.W. winds, and in the reverse direction during the winter or with a prevalence of strong N.W. winds, at the rate of ½ to 1½ knot per hour.

Owing to the great stretch of ocean to the west, very severe storms are possible, and the south-west corner of Australia, Cape Leeuwin, has a bad name for them; but the resulting waves have not as much effect on the shore generally as would be the case if the sandy beaches did not exist. The winds on the west coast between North-West Cape and Cape Leeuwin are generally from some southern point, mostly between S.S.W. and S.S.E. During the summer they blow almost constantly from this quarter, but their regularity is interrupted in winter by occasional winds between north and west, which at times blow with considerable violence.

The sandy country, the small flow of the rivers, and the small

range of the tides, combined with strong winds and currents, have not only deprived south-western Australia of natural harbours, but have also closed all the rivers to navigation. Thus, taking the rivers from north to south, the mouths of the Greenough and Irwin are generally closed, the Arrowsmith does not reach the sea, and the Moore is too shallow for boats of more than 6 feet draught. The Swan was blocked by a bar, and the Murray has a bar with seldom more than 6 feet of water over it. The Leschenault inlet has a shallow sand bar across the entrance with seldom water enough for a boat; the common mouth of Wonnerup inlet and Vasse estuary is often closed entirely; and the Blackwood river has a bar with 4 or 5 feet of water in summer, and 6 or 7 feet in winter and after heavy freshets.

As in the somewhat analogous case of the west coast of the United States of America, the natural forces—winds and waves, tides and currents—although acting in the same directions all along the coast, have not produced the same results at all the rivers, owing, no doubt, to differences in the trend of the coast and to the interposition of such natural features as headlands. Thus the Leschenault and Swan, being protected from the south-west, have turned their mouths, or had them forced, permanently southward; while the Murray bar, on the other hand, when cut through by freshets, forms again from the south, the mouth of the river being forced farther and farther north, until finally closed.

Moreover, there are manifest signs in places that sand-drift, travel, and accumulation are still in progress. Thus, at Hamelin Bay the sand-drift has been gradually moving eastward, maintaining apparently a height of 150 feet on its eastern side; for, a few years back, the decayed tops of karri trees, just showing through the sand, were about the same height as those not yet enclosed: and at Busselton the coast-line has been moving steadily outwards, a long length of the old timber jetty having been buried in sand to the decking. Nevertheless, the extent to which the various bays and rivers are affected by continuous sand-movement, or even whether some are not so affected, has not been easily judged in the absence of continued observations; and, as will appear farther on, it has been exceedingly difficult for responsible engineers to advise with confidence.

THE ALBANY AND BUNBURY HARBOURS.

The Albany or Princess Royal harbour is situated in the north-west of King George Sound on the southern coast, and is almost entirely landlocked. The circumvallation consists of granite peaks

connected by ridges of sandy strata, and the entrance is narrow. Although sand-drift and travel have, in the course of time, filled the large enclosed basin, there is not now much apparent movement of the sand, nor does any river flow into the harbour.

The works necessary and carried out from time to time to accommodate the trade of the port have therefore been comparatively simple, and have chiefly comprised dredging to widen and deepen the entrance-channel, to provide a suitable anchorage-basin, and to afford access to the timber jetties; these, with their accessory goods-sheds and railway-lines, a couple of lighthouses, and some beacons, complete the works executed.

The anchorage-area is now 350 acres, with 30 feet of depth at low water, and the channel to the town jetty is 400 feet wide and 23 feet deep. There is a swinging-basin abreast of this jetty. Up to September, 1903, about 3,225,000 cubic yards of material, consisting of sand, silt, shell, and decomposed or half-formed rock had been removed and deposited at sea.

The Bunbury harbour-works consist of three main items: a lighthouse on a commanding hill, a jetty jutting out into Koom-bana Bay, and a protecting breakwater.

The westernmost of the archæan belts previously mentioned is still in evidence from Cape Leeuwin to Cape Naturaliste, north of which lies Geographe Bay which, as already stated, shows marked evidence of coast-advancement. To the north of this bay the continuity of the coast-line is broken by the small indentation of Koombana Bay. This has resulted from a basaltic outcrop and overflow, which, while affording protection from the south-west, left the combined mouths of the Collie and Preston rivers exposed to the lesser but still important weather from the north-west. Although these streams with their tributaries drain an area of, roughly, 2,000 square miles, having an average rainfall of 31 inches per annum, their discharge, for reasons already stated, is too small to counterbalance the effect of the sand-accumulation caused by winds and waves. The mouth of these rivers, therefore, has been gradually forced southward and now lies under the very lee of the basaltic headland, the estuary being exceedingly shallow.

The small town of Bunbury is the natural port for a large district, but, the river not being navigable, it was not possible, in the earlier days of small things, to provide more in the way of harbour facilities than a naked jetty projecting into the bay. In 1893 the head of this structure, although in only 14 feet of water, stood 2,178 feet from the shore, exposed to every weather north of due west. So soon, therefore, as the increasing volume of trade brought

about the use of coasting and other vessels of deeper draught, the question of a better-protected harbour was forcibly brought to the front, especially when actual shipwreck occurred.

The physical features of the port clearly lent themselves to the formulation of two differing harbour-schemes, and two such schemes were submitted accordingly, in September, 1896. One was for an inner harbour, and the second, which was finally adopted, was for a curved submerged breakwater, to have an ultimate length of 6,000 feet, in extension of the basaltic headland, thus forming an outer harbour. The breakwater, which was ultimately built as a visible and not as a submerged structure, consists of tipped rubble stone. Shoaling occurred in the basin and at the outer end of the breakwater, but it is hoped that a further extension will cause the latter shoal to be scoured away by natural means.

FREMANTLE HARBOUR: LOCALITY AND PRELIMINARY SCHEMES.

Attention has been drawn above to the evidence of sand-drift, travel, and aggregation at Hamelin Bay, at Busselton, and at Bunbury. North of the last-named there is for about 50 miles a succession of long shallow lakes or lagoons, separated from the sea by the sand-hills or dunes before referred to. The northernmost alone of these lakes discharges into the sea, the opening being maintained by the River Murray, which in heavy freshets cuts through the sand bar at Mandurah. The bar, however, regularly forms again, the mouth of the channel being forced more and more to the north until closed.

A few miles north of Mandurah there commence two parallel lines of reefs and islands, which bear N.N.W. and terminate in Rottnest Island. The deep gulf enclosed between the chains of reefs and islands and the mainland is termed Cockburn Sound, the northern portion being Gage Roads, into which falls the Swan River through an estuary (Fig. 1, Plate 7) which is tidal for about 20 miles. Although there is marked evidence, in the spits to the east of the various islands, of previous sand-travel, there is nothing now evident to show that there is present continuous growth of spits or shore—at least in positions to which resulting inconvenience would direct attention. Nevertheless, the possibility of silting of a harbour or its entrance, whether through continuous sand-travel or through local movement to and fro, has been a source of anxiety to every engineer who has had to deal with the design of proposed harbour-works in this locality.

There is ample depth over a large area and close to the shore in the sheltered waters at the head of Cockburn Sound. There are also, between the islands, channels of approach, which are capable of improvement and even now are used by vessels of moderate draught loading timber at the Rockingham jetty. But the seaport of Fremantle, small in the days before the expansion of the colony due to the gold-discoveries, was located at the mouth of the Swan River, Perth, the capital of the State, lying 12 miles up the river; and it naturally resulted that any new harbour-work must be built in the immediate vicinity of the mouth.

All that had been achieved by 1875 was the construction of three small jetties, one in the river in 5 feet of water, and two just south of Arthur Head, in 8 and 12 feet of water respectively. The bar at the mouth of the river prevented it from being used for navigation, while any jetty advanced beyond the shelter of Arthur Head became subject to the full force of the north-westerly weather. One of the two jetties shown was so advanced and improved from time to time, but the facilities obtained were not to be compared with those of the inner harbour since constructed, and the jetty is no longer in use.

The importance of obtaining a sheltered harbour was recognized, however, even in those early days, and notwithstanding the paucity of population and revenue—indicated in the following Table—the question was entered into with vigour.

PROGRESS OF WESTERN AUSTRALIA.

Year.	Population of whole Colony.	Revenue of whole Colony.	Railways in Colony.
	No.	£	Miles.
1829	Beginning of colonization.		
1875	26,709	157,775	Nil.
1877	27,838	165,413	Nil.
1887	42,488	377,904	168
1891	53,279	497,670	203
1902	215,157	3,690,585	1,360

Fortunately for the coming port, no minor work was put in hand, and in 1877 the matter was placed before the late Sir John Coode, K.C.M.G., Past-President of The Institution, who has left it on record that the various schemes of improvement proposed up to that date might be “briefly summarized as follows:—1st, works contemplating the provision of the required accommodation elsewhere than

at Fremantle; 2nd, works for the improvement of the river lying between its debouchure and the bridge which carries the road to Perth; 3rd, external works running from the shore into deep water; 4th, open-piled jetties; 5th, solid sheltering breakwaters detached from the mainland." It is interesting to note that the germs of his own proposals, made later, are contained under the fifth heading, and those of Mr. O'Connor, made later still, under the second.

Sir John Coode made two reports; one in November, 1877, based on papers submitted to him, and the other in March, 1887, after a few weeks' stay in the Colony. His reports show that the Colonial authorities were pressing for accommodation much beyond the needs of the time, and on the other hand, as an engineer responsible to the Crown authorities, he could not have been otherwise than struck and hampered by the fact that, compared with other colonies, Western Australia had made such small progress that, 60 years after the foundation of the Colony, the population was but little over 40,000, while the revenue was so small that the interest on any considerable capital expenditure would be a serious matter.

Sir John Coode, too, had evidently been given to understand that there was decided evidence of sand-travel. Those were days before the introduction of the suction sand-dredger, and, as already mentioned, the alleged certainty, or even the risk, of large recurring expenditure could not be incurred. His proposals, it will be seen, allowed for movement of sand along the foreshore, and so far they agree with those he submitted for Timaru, in New Zealand, where the shingle-travel was the largest he had met with up to that date. There the accumulation was subsequently found to be 80,000 to 100,000 cubic yards per annum, and later on he recommended that it should be dealt with by means of dredgers.

It would have been very interesting to see how his proposals for Fremantle would have been modified after the advent of an efficient suction-dredger, after the grant of responsible Government, and after the sudden increase of population and resources which a few years brought about. Even as they stand, the proposals demand reference in detail.

Another local belief was that storms in the locality were very violent, and—what is difficult to understand, considering the coast and depths in the offing—dread in effects. The Author reads Sir John Coode's proposals as showing that he did not consider wind and wave would be severe in effect, except where the works were broadside on to the direction of the worst weather. It is also clear that Mr. O'Connor, who reported later, was of a similar opinion. Nevertheless, the belief as to the severity of the storms in this locality

persisted for many years, and forced the Government later on to incur needless expenditure in order to induce the English mail-steamers to call at Fremantle when the new harbour was in a suitable condition to receive them.

In 1877 Sir John Coode's estimate for advisable works was £242,000, and writing later of these works, he said :—

"I arrived at the conclusion that the form of work at that time best suited to fulfil the conditions of the case would consist of an open timber viaduct 1,800 feet in length, extending from Arthur Head in the direction of the Beagle Rocks, terminating with a T shaped solid pier, the two arms of which would be of the aggregate length of 1,500 feet. In addition to the foregoing, I proposed a spur jetty of 300 feet in length, to be formed at the junction of the viaduct with the solid pier, so that the total amount of berthage under this project would have been 1,800 feet in a depth of 20 to 27 feet at low water."

Reporting in 1887, after a flying visit to the State on his way home from Eastern Australia, and at a period of the year when personal observation was impossible, Sir John Coode said :—

"I had previously considered that these shoals [the Success and Parmelia banks, Fig 1, Plate 7], which are of remarkable shape, were merely accumulations of sand, and that their existence afforded strong if not conclusive evidence of sand travel to the southward and of the shoaling which would consequently result from the construction of any solid works extending directly from the shore. . . . I subsequently framed a further memorandum . . . requesting that borings might be made through the Success and Parmelia banks, in order that their character might be clearly and unmistakably ascertained. . . . From these additional particulars it is now clear that the banks referred to are accumulations of sand, and although the records of depth taken from time to time by the Admiralty authorities are not sufficiently numerous to enable their growth to be clearly traced and defined, nevertheless sufficient data are at hand to show that in all probability they are fed by the preponderating southerly movement of sand through Gage Roads and along the coast.

"Looking at the results of the supplementary borings on the Success and Parmelia banks, it would not be prudent to construct any solid structure in direct connection with the shore. To provide for the unimpeded movement of the sand, it will be requisite that any sheltering work at Fremantle should be detached from the mainland, the connection with the shore being effected by means of an open viaduct so arranged as to admit of the unrestricted passage of the sand without causing its deposition."

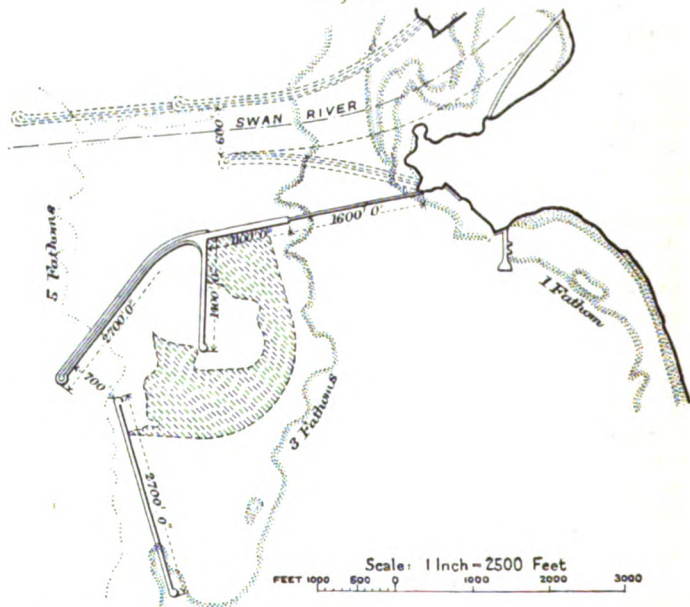
Works in Cockburn Sound being precluded, partly because of the vested interests of Perth and Fremantle, and partly owing to the alleged sand-travel, and an inner harbour in the river with an outer harbour formed by solid works connected with the shore being undesirable also on account of the alleged sand-travel, there remained only the alternative of an outer harbour connected with the shore by an open viaduct. The works of such an outer harbour would of necessity be broadside on to the worst weather.

In this connection, therefore, Sir John Coode wrote:—

“Gage Roads are open to the full stroke of the seas from N. to N.W. . . . The heaviest gales commence from N. to N.W. and travel westerly to about S.W., by which time they usually abate. These gales bring in heavy seas. . . . Whilst at Fremantle I had an opportunity of witnessing the effect upon the sea in Gage Roads of a very strong gale from the N.W. . . . This occurrence was so far fortunate as it enabled me to form a correct idea of the wave stroke that would fall on artificial works in a gale of unusual severity. . . . Having now had an opportunity of witnessing the force of the sea which would have to be encountered by these works, I am satisfied that nothing less substantial than structures of solid concrete would meet the conditions of the case in a permanent and satisfactory manner.”

Sir John Coode's proposals, which, as previously stated, are of the greatest possible interest, were for works to be constructed in two successive portions. The first portion was for the accommodation

Fig. 2.



of the trade of the time and a few years ahead, carried on in ordinary trading vessels, and the second portion was for the time when larger boats, such as the Peninsular and Oriental and the Orient mail-steamers, might be expected to visit Fremantle.

The first portion of the works, estimated to cost £495,000, was to have consisted of a root or abutment with sites for offices, warehouses, etc.; an open viaduct 1,600 feet long (Fig. 2), carrying two

lines of railway; and a right-angled concrete pier 37 feet wide, the arms of which would have extended 1,100 feet in continuation of the viaduct and 1,400 feet at right angles thereto, in a southerly direction. Berthage to the extent of 2,500 feet was to have been formed on the lee side of the two arms of the concrete pier, in depths, after dredging (included in the estimate), of 26 feet to 29 feet at low water.

The second portion of the works was to have consisted of two sheltering breakwaters, each 2,700 feet long, to be built of riprap with a capping of concrete suitably arranged. Their cost was put at £101 per lineal foot, or a total of £545,000.

The whole cost, therefore, of the accommodation for vessels of all classes was estimated at £1,040,000.

Nothing practical, however, ensued until, some years later, the late Mr. C. Y. O'Connor, M. Inst. C.E., was appointed Engineer-in-Chief. Applying himself particularly to the question of sand-travel, he came, after much personal observation, to the conclusion, and subsequently so stated, that the information supplied to the engineers who had reported previously was in his opinion erroneous; that he could not himself find evidence of sand-travel; but that, if it did exist, it could be counteracted by dredging in the manner that Sir John Coode had recommended in the case of Timaru, where, Sir John had estimated, the expenditure of £4,000 to £6,000 per annum would overcome the evil.

Such being his conclusions regarding sand-travel, and the Colony, by now granted responsible Government, having also progressed and being mistress of her own finances, Mr. O'Connor was able to submit a more comprehensive scheme, and decided that it would be best to open up the river. There was a choice between two methods of doing this; either through the bar at the mouth of the Swan, or by means of a channel through the narrow neck of land opposite Rocky Bay (Fig. 1, Plate 7).

The latter would have permitted comparatively easy access to Perth, and a comparatively easily-dredged basin of good size for ships to lie in would have been obtained. It is interesting, therefore, to note the reasons why Mr. O'Connor did not favour the Rocky Bay channel. In a report dated the 21st December, 1891, he said:—

“With regard to the Rocky Bay scheme, it has no doubt some advantages over an entrance at the mouth of the river, as it would be somewhat cheaper to construct, and has clearer exit into bold water. The navigation, however (unless an immense amount of dredging were done), from thence to Fremantle, would be somewhat tortuous, and the project has not therefore, I think, on the whole, sufficient advantages to warrant the creation of new interests, and the sacrifice of existing ones, to which it would no doubt have a tendency.

"If the railway had not to be crossed twice, and the main road also twice, the state of the case would be materially different; but the difficulty and expense which would be entailed by these four crossings (or by deviations of road and railway sufficient to obviate them), considerably counterbalance the other advantages of the Rocky Bay entrance.

THE SCHEME DECIDED ON.

Choice therefore fell on the plan of opening the bar and constructing a harbour at the mouth of the river. The scheme, approved by Parliament early in 1892, was estimated to cost £800,000 for the bare harbour, exclusive of all accessories.

Fortunately, however, the continued progress of the Colony rendered it necessary that the scheme should be enlarged, and the cost to 1903, soon after which the Author left the State, was actually £1,458,940, made up in the manner indicated on p. 180.

The harbour constructed for this sum of money is in the mouth of the Swan River at the end of Gage Roads (Fig. 3, Plate 7). A main and far-showing light, high on Rottnest Island (Fig. 1, Plate 7) indicates the locality; a second and minor one at Bathurst Point, also on this island, enables it to be safely rounded; and a third, at Woodman's Point, directs vessels along the roads until the lights on the breakwater are opened. Between the moles a channel, 30 feet deep at low water, leads to a basin of the same depth, bordered on north and south by quays.

On the south the basin is well protected naturally by high ground, but on the north the narrow neck of land, more than once threatened and once breached by the sea, has been widened out, protected on its sea face by stone revetting, and provided with an artificial breakwind. On this side of the basin, the widening and reclamation have provided a considerable area of land, and the quay is laid with railway-lines connected directly with the railway system of the State.¹ On the south side reclamation has also provided a large expanse of land behind the quay, and partly on this ground are situated the sorting-sheds for cargo, and behind them the railway sorting-yards, in direct communication with the railway goods-shed and main station.

It was the Author's ambition that the harbour should receive an increasingly valuable endowment in the shape of a line of stores and bonded warehouses situated between the sorting-sheds and sorting-yards; but the idea does not appear to have been persevered in.

¹ The plan of the harbour (Fig. 3, Plate 7) refers to 1903, and the arrangement of the railway-lines on the south side was subject to alteration later.—C. S. R. P.

THE NORTH AND SOUTH MOLES.

The two moles or breakwaters which flank the entrance-channel (Fig. 3, Plate 7) are of unequal importance, as from the situation of the south mole the action of the sea on it can be relatively only small. The north mole, on the contrary, protects both the harbour-entrance and its fellow breakwater from the severest weather. In Fig. 5 are shown the cross sections of the two moles. The south mole remains practically of the length originally proposed in 1892, but the north mole was extended first of all by about 500 feet to 3,450 feet in length, and then by a further 1,350 feet.

With the exception of the parapet on the north mole, both breakwaters were built entirely of tipped stone, the stones being allowed to take their own slope; end-tip trucks were used to form the leading roads, and side-tip trucks for widening out. It was proposed in 1897 that the extension of the north mole by 1,350 feet should be a submerged breakwater, but it was eventually built to full height. Also, it was feared at first that this extension of the north mole might have to be of solid concrete, on account of the probable seas; but the success of the riprap system in the older portion determined the employment of this cheaper method in the extension also.

The cross section aimed at—that is to say, the slopes to which it was anticipated the sea might eventually draw the stone out—is shown in Fig. 5; and in order that there might be material enough for the flat slopes, a parapet, as shown, was tipped to the following approximate top widths of mole: south mole, 25 feet at the root, and 40 feet at the commencement of the round head, which was given a diameter of 80 feet; north mole, 30 feet at the root, 44 feet at 2,300 feet out, 52 feet at 3,300 feet, and correspondingly wider farther out. The rounded head at the inner termination was given a diameter of 130 feet, but that at the extreme end was made only 80 feet in diameter, which the Author determined would be sufficient, in view of the fact that better stone was then being used.

The stone used in the moles was of two kinds. From the levelling down of a portion of Arthur Head and from the quarry at Rocky Bay there was obtained a material which commenced by being an arenaceous limestone, or a calcareous sandstone, with a specific gravity of 2.64, which, according to Mr. O'Connor's calculations, would have produced a mole weighing 1.23 ton per cubic yard. But, as the Rocky Bay quarry developed, the quality of the

stone deteriorated ; it was found to be both lighter and considerably more friable, and recourse was had to a new quarry at Boya, some distance inland. This yielded granite of good quality with a specific gravity of 2·73.

Work on the north mole was commenced in November, 1892, with stone from Rocky Bay, and it was completed as far as the 3,450-foot mark in November, 1895. The south mole was commenced in August, 1894, with stone from the levelling down of Arthur Head, but progressed very slowly until, on cessation of work on the north mole in November, 1895, the stone-trains from Rocky Bay were diverted to it, thus enabling completion to be effected in August, 1897. The extension of the north mole was commenced in July, 1899, and completed in December, 1902.

The south mole was constructed entirely of the limestone and sandstone described, and from completion in August, 1897, until the latter part of 1902, no repairs were required. In this year, in preparation for the permanent light-tower shortly to be erected, the extreme end—which had worn and subsided slightly—and the adjacent 150 feet were topped and faced on each side with granite.

The first portion (i.e. 3,450 feet) of the north mole, including the parapet, was constructed of stone of the same class as was used for the south mole, and so also was about three-fourths of the extension. But for the outermost portion it was considered better to use granite, and in 1902 the sandstone portion was given a coating of granite on the sea face.

During construction of this mole there was considerable loss and waste of the sandstone at the tip-head in stormy weather, the work advancing very slowly in the winter months. In one bad month in 1901 there was a loss equivalent to 2 per cent. of the whole contents of the mole between the 3,450-foot and the 4,500-foot marks. But, once the stone had settled into position, little occurred. Thus, for 2 years after completion of the parapet, on the original length of 3,450 feet of the north mole, only some slight flattening of the slopes took place. During the winter of 1898, notwithstanding several severe storms, only 6 per cent. of the reserve stone in the parapet was drawn into the sea slope. The expenditure on repairs was £700 in 1898–99, and thereafter no further outlay was incurred to December, 1900. It may indeed be said that only the anticipated clawing down of the slopes by the sea occurred, and the extent even of this was moderate, as is indicated by Figs. 6, Plate 7.

DREDGING.

The entrance-channel of the harbour, shown in Fig. 3, Plate 7, is 450 feet wide for a length of 3,000 feet east of the 30-foot contour, and it then gradually widens out to 575 feet in the next 1,550 feet of length, where it may be said that the basin commences. From this point the widening continues until, at a further 1,800 feet, the full basin-width of 1,400 feet is attained. This width is maintained for 2,350 feet, and the basin was designed to narrow thereafter to the railway-bridge, which is 10,000 feet east of the 30-foot contour-line.

The cutting-down of the area shown as dredged, almost all to a depth of 30 feet below extreme low water, entailed the dredging of about 10 million cubic yards of material, exclusive of the quantity above water-level which was excavated in the dry and carted into the area to be reclaimed. Of the dredgings, about 2 million cubic yards consisted of rock, and the balance of sand and silt, much of which was too compact to be worked economically with a suction dredger.

By far the larger portion of the rock to be excavated was in the entrance-channel and the adjoining portion of the basin. Here, blocking the entrance, was the bar, consisting of a long rolling ridge of rock, principally coralline limestone and sandstone, which just across the mouth of the river showed a broad crest rising to low-water level (Fig. 4, Plate 7). Of the rock about three-fourths was drilled and blasted before being removed by bucket dredgers, and of the sand and silt about 30 per cent. was excavated by bucket dredgers and the balance by suction dredgers.

The engineering problem, therefore, was twofold: first, how to bring both bucket and suction dredgers quickly to work; and, secondly, how to dispose of the dredgings economically and to advantage. To these more strictly engineering conditions was added the imperative necessity of rendering portions of the harbour available from time to time for the rapidly advancing trade of the State.

The first requirement, therefore, was to cut a channel across the bar, and drilling- and blasting-operations were commenced in July, 1894. On the 30th of the previous month the north mole tip-head stood at 2,300 feet from the land end at Rous Head, so there was sufficient shelter from the direction of the worst weather. The drilling and blasting were conducted from stages, as explained later, and successive strips 100 feet or so in width were drilled and blasted to a depth of 15 feet below low water. This depth was completed in October, 1895, and the drilling and blasting of the northern half (225 feet) of the channel followed immediately. It was effected to

the full depth of 30 feet at first and the work progressed rapidly, but complaints being received that the heavy charges necessary were damaging buildings in Fremantle, the drilling and blasting had to be carried out in two lifts, the progress being slower and the cost enhanced. When this section was completed, attention was turned to the southern half of the channel.

Some of the ground had to be gone over twice where the rock bar was exceptionally hard, so as to save the dredgers. On the other hand, in the later stages of the work, when the northern half of the channel was in constant use by incoming and outgoing vessels, drilling and blasting were dispensed with, for fear of damage to the vessels, and the dredger was kept hard up against the rock to be removed, scraping away until the channel was entirely clear.

Dredging was practically started with a bucket dredger, the "Fremantle," which arrived from England in October, 1894, and commenced work at the seaward edge of the bar on the rock already blasted. By September, 1895, a channel 200 feet wide and 12 feet deep at low water had been cut through the bar, and the "Fremantle" passed into the river. She was followed, in January, 1896, by the suction dredger "Premier," which set to work on the sand in the inner basin. To these were subsequently added a second bucket dredger and a second suction dredger, all four vessels designed by Messrs. Coode, Son and Matthews, and built, the bucket dredgers in Scotland and the others in Holland. A tug and barges were also used, for a part of the time, to hasten the progress of the work.

In May, 1897, the S.S. "Sultan," an Australian coaster, made the first passage over the bar, the channel being 180 feet wide and 20 feet deep at low water. In October, 1897, the S.S. "Cornwall," an open-sea vessel, 420 feet long and of 5,480 tons burden, berthed in the river. By July, 1899, the German mail-boats successfully used the harbour, swinging in a basin then only 650 feet in width; and in July, 1900, the dredging was sufficiently advanced for the mail-steamers of the English lines to be induced to call regularly.

Of the dredgings, a small quantity of suitable stone was deposited on the line of the north mole, and about 3 per cent. was pumped by one of the suction dredgers on to the area to be reclaimed, still leaving nearly $9\frac{1}{2}$ million cubic yards to be disposed of. It was decided that the most suitable dumping-ground for the dredgers and barges was along, and not far from, the shore north of the north mole. Contours of the ocean-bed adjoining the river-mouth taken in 1897 disclosed the proximity of the 2-fathom contour to the shore north of the river. This was dangerous; for a breach, in storms, of the narrow isthmus connecting Rous Head with the mainland (Fig. 1,

Plate 7) had been threatened more than once, and had occurred once. Dumping of the dredgings was commenced, therefore, near the north mole and continued farther afield as each patch of ground was rendered too shallow for use. Ultimately the spoil was spread over an area bounded on the south by the north mole, on the west and north-west by the 3-fathom contour of 1902, on the south-east by the shore (as near, that is, as the dredgers and barges could get), and on the north-east by a line in continuation of River Street.

Of the whole quantity dredged and deposited in the years 1893-1901, namely 8,321,000 cubic yards, about $63\frac{1}{2}$ per cent. remained within the area of deposition, and a further 17 per cent. was redeposited by the sea outside this area but within the limits of the soundings: of the balance, 113,000 cubic yards, or $1\frac{1}{2}$ per cent., went to advance the foreshore, and the remaining 18 per cent. was entirely lost. An analysis of the weather-conditions in three periods from 1899 to 1901 shows that there was some correspondence between the weather and the percentage of dredgings drawn out to sea and lost to ken. These lost dredgings were not carried to the south of the harbour so far as the soundings showed, for comparison of the 1893 and 1903 contours disclosed the fact that south of the entrance-channel there had been practically no change in the ocean-bed. Even the patch south-west of the end of the south mole had remained in the same condition throughout the 10 years.

North of the north mole there have, of course, been changes, the foreshore and the 1-, 2-, 3-, 4- and 5-fathom contours all having advanced; but this, as already shown, can be ascribed to the sea having moved and distributed the dredgings. Between the surveys of 1902 and 1903 the 5-fathom contour due north of the north mole advanced markedly, and, on the other hand, the survey of 1903 also disclosed (Fig. 3, Plate 7) that the water opposite the narrow isthmus connecting Rous Head with the mainland was deepening and the foreshore was receding in this locality.

Within the harbour, however, and in the entrance-channel there had been very little silting, what there was, moreover, being counter-balanced by considerable scour in other parts of the inner basin. This area and a large portion of the entrance-channel were cleaned up between April and October, 1904, by a bucket dredger. The silt patches, also shown in Fig. 3, which formed by December, 1905, aggregated only 1,500 cubic yards, reckoning above the minimum harbour-depth of 30 feet below low water. As the sounding-line showed in the same month scour to depths of 40 feet below low water, it is probable that these silted patches are due to purely local causes.

RECLAMATION.

On each side of the basin was a considerable area of land which required to be cut down or filled up before it could be utilized for sheds and sidings, and early attention was given to this work of reclamation. The general levels fixed on were 10 feet above low water on the south of the harbour, and 13 feet on the north. The slopes of the earthwork were designed to extend below the timber wharves, and to have a protective facing of dry stone (Fig. 7, Plate 7) reaching down to the full depth of the basin. On the north, which was taken in hand at leisure, and where the filling consisted entirely of tipped material, the protective water face of stone was easily obtained; the stone was tipped on the top of the bank and gradually sank down, and was replaced at the top as a suction dredger excavated the basin to its full depth. The bank was coated to the bottom, and when the sloping protection had taken its set it was finished off at the top with dressed stone laid dry, in order to prevent nuisance from matter which might lie in the interstices of rough stone (Fig. 8, Plate 7).

On the south the reclamation had to be pressed forward, and the best method of construction was subordinated to a considerable extent to the exigencies of trade: also, the filling consisted partly of dredgings pumped on to the land, and partly of waste from the stone-quarry brought down in trucks. It was necessary, therefore, to provide a retaining-bank for the dredgings, with openings through which the water could drain out rapidly. The first work, consequently, was the construction of a bank from end to end, along the edge of the reclamation-area, 10 feet above water-level, and 20 feet wide. This bank had also to serve as the formation for an independent railway-line for the stone-trains from the Rocky Bay quarry to the south mole, and it was so used as soon as the railway-bridge across the Swan was completed and the north mole had reached its intermediate terminus.

The bank was faced on the harbour side with tipped stone. This would have slipped down as the dredging progressed, and a coating would easily have been obtained as on the north side of the harbour, but the work was rendered difficult, as the dredging could not be carried out fast enough for the circumstances of the case. Thus, orders were issued by the Government at the end of August, 1896, to commence wharf-construction, and by the 30th June, 1897, about 1,500 feet of quay was available for use by vessels, though the depth of water alongside was only 20 feet. By the 30th June in the following year the reclamation on the south side was completed,

and 4,450 feet of the south quay was available for use but the dredging was not complete adjoining any portion of the quay; the actual conditions were, 1,850 feet of wharf with 25 feet depth of water, 1,000 feet with 20 feet of water, and 1,600 feet with 12 to 15 feet. The result was that both the material below water and the excess stone above and below water remained badly disposed, as shown in Fig. 7, Plate 7. Matters were improved somewhat by the use of a small suction dredger to excavate under the wharf, but only part of the material could thus be moved, and it was not possible to employ a more powerful tool, for fear of damaging the ground below the sheds and a temporary passenger-station erected adjoining the quay.

The irregular disposition of the stone was not only a source of expense when the quay, constructed under pressure to accommodate traffic, was widened out to the present width, but was also a cause of serious nuisance, as the garbage which collected on the rough surface assisted in furnishing food for colonies of rats that lived in the interstices of the stonework; and these rats were a considerable source of danger in times of bubonic plague. In order to obviate these difficulties, the stonework, from the top to as low as the water would permit, was roughly "napped" to a face and coated with concrete, as shown in Fig. 9, Plate 7.

WHARFAGE.

The wharves or quays, which are of timber on timber-pile substructures, have an aggregate length of 9,255 feet, distributed thus (Fig. 3, Plate 7): north mole, 1,000 feet; south mole, 300 feet; jetty on the north side of the harbour, 900 feet on both sides; north quay, 2,000 feet; south or Victoria quay, 5,055 feet.

The wharves on the moles, which were built in 1897 to meet the great rush of traffic due to the rapid expansion of the State, are noteworthy because the piles of the substructure were driven into the moles, a fact which illustrates the softness of the stone and the hardness of the jarrah piles. These wharves have stood and answered their purpose well, notwithstanding the comparatively exposed position.

The jetty, which projects into the harbour from the north quay, was completed in 1900. It forms a part of the arrangements made to induce the vessels of the British-Australian mail-service to call at Fremantle, and it is a direct consequence of the mistaken ideas that prevailed in some quarters regarding the weather on this part of the Australian coast. It was feared that vessels moored against the south quay might not be able to get away safely to time in spells of bad weather. The Author believes the jetty has never

been used for the purpose it was built to serve, the mail-steamers having always found accommodation at the south quay, or having in time of bubonic plague remained at moorings in the centre of the harbour.

Of the two main wharves, that on the north of the harbour is one of the items which were designed while the Author was in immediate charge of the harbour-construction. The work in the wharf was straightforward; the piles, which are 12 feet apart from centre to centre each way, were steam-driven after the dredging was completed, and have successfully carried the loads placed on the wharf.

The history of the remaining wharf, the south or Victoria quay, in contrast to that of the other four, has been a troubled one, due partly, as previously stated, to efforts being made to provide harbour accommodation while the works were in an inchoate condition, and partly to divided counsels. Mr. O'Connor's proposals were framed to provide for easy transit inland of goods so destined, combined with as easy discharge of merchandise intended for Fremantle; while the Author hoped, by institution of free and bonded stores behind the quay, not only to assist in rendering Fremantle a great distributing-centre, but also to provide an endowment which would assist the managing authority to make of it a very cheap port indeed.

Mr. O'Connor desired the wharf and reclamation to be on the same level, with four lines of rails and hydraulic cranes on the former, to facilitate handling and distribution of cargo. As a compromise, two lines only were placed on the wharf, the cranes were dispensed with, and sorting-sheds were built on the quay, which was raised to 3 feet above reclamation-level in order to permit of easy delivery into carts conveying merchandise into Fremantle. New firms, however, not finding convenient situations in Fremantle, built their warehouses in Perth, the chief town of the State; and the older firms have had to follow the example set them. After the Author had left the State, he recommended the installation of electric cranes. These have been put in operation and have proved fully the advantage of equipping such a wharf with cranes; but the arrangements cannot be so satisfactory as if they had formed part of a complete original scheme.

In Fig. 9, Plate 7, is shown in plan the resulting quay, with its sorting- or transit-sheds existing and proposed. The westernmost 2,000 feet or so of the quay was built to 10 feet above low-water level, and of the section shown in Fig. 7. It was subsequently widened. After the Author had left the State, the raising of this portion to 13 feet above low-water level was sanctioned, and a portion of it was raised. It was this 2,000 feet that was

hurriedly built, as already mentioned, to accommodate traffic; it sank to some extent, and advantage was taken during the raising to strengthen the wharf by inserting extra piles.

MINOR WORKS.

Of the minor works, three deserve some notice, namely, the slipway, the breakwind, and the lighthouses.

The slipway is situated at the root of the north mole (Fig. 3, Plate 7), and was intended to take vessels of a maximum dead weight of 650 tons and a maximum length of keel of 160 feet. It is a timber structure, the cradle being 185 feet long; and at ordinary spring-tides the depth of water over the keel-blocks, with the cradle at its lowest, is 10 feet 9 inches forward and 18 feet 6 inches aft. It was thus able to take any of the four dredgers used, which otherwise could not have received attention nearer than Melbourne, a good fortnight's journey, to and fro. The slip has also been of the greatest use to the smaller coasting craft.

The breakwind was erected to protect the basin from north-westerly gales, and it extends eastward from the root of the north mole for 2,200 feet along the low-lying isthmus north of the harbour (Fig. 3). It consists of quarry-refuse faced with roughly-dressed sandstone, laid dry, which is surmounted by an open picket fence consisting of 3-inch wooden uprights spaced 6 inches apart from centre to centre; the top is 45 feet above low-water level.

The five lights connected with the harbour were all obtained under the advice of Mr. W. T. Douglass, M. Inst. C.E. The main light, situated on Rottnest Island, is installed in a stone tower, and is 264 feet above sea-level at the focal plane. It is of the first order, dioptric, and with a single flash every 20 seconds. The second light, which enables Rottnest Island to be safely cleared, is situated at Bathurst Point. It is a fixed second-order dioptric light, 98 feet above low water at the focal plane. The third, or Woodman's Point light, directs vessels up Gage Roads until they open the harbour-entrance, where, at the end of each mole, there is a cast-iron tower 20 feet high, carrying a fourth-order fixed light.

There was some discussion as to the manner in which vessels should be directed up to the harbour, and at first it was proposed to erect two lights, one on Fish Rocks and the other at Woodman's Point. But it was considered that a cheaper and also better arrangement would be a single light at Woodman's Point throwing sectors of coloured light. That installed is fixed, of the first order,

and occulting, with its focal plane 126 feet above low-water level. The sectors, on the advice of Captain Russell, then Chief Harbour-Master of Western Australia, were so arranged that the westernmost or red sector can be used, if they so desire, by vessels of small draught, and the centre white portion by vessels of deeper draught, while the easternmost or green sector indicates dangerous ground.

DETAILS OF COST.

Almost the whole of the work was carried out departmentally, and from the detailed accounts kept of expenditure and of work done the following particulars have been abstracted regarding the excavation of the channel and basin, which is divisible into two parts: first, the drilling and blasting of the rock, and secondly, the dredging.

It was essential that the drilling and blasting should be started quickly and pressed on with; to obtain special plant from Europe would have meant delay, and floating drilling-craft could not be satisfactorily employed, owing to the shallowness of the water over a large portion of the bar-crest and other parts; moreover, this rock was soft enough for the hand-drill, which was therefore employed and worked from stages. These consisted of light 30-foot planks carried on four-footed trestles with ladder-shaped tops, permitting of the planks being raised or lowered. In a couple of days 20,000 square feet of this staging could be placed in position, and it could carry 120 to 160 men, drilling with regularity in 20 feet and more of water. The holes were 8 feet to 12 feet apart, according to the quality of the rock, and the charge consisted of 12 to 15 lbs. of dynamite or gelignite, a ton of explosive sufficing for about 5,600 cubic yards of rock blasted. The expenditure on the various sub-heads of the work varied from time to time within limits, but approximately the total cost can be subdivided thus:—

	Per Cent.
Stages and other plant	16
Placing stages	6
Tugs, punts, pumping, etc.	4
Repairs to various plant	4
Stores, piping, drills and other tools	6
Explosives and blasting	16
Drilling	48
Total	100

The material dredged consisted in various parts of rock, blasted and unblasted, and of clay and sand; most of the sand was removed by the suction dredgers, but large quantities were excavated by the bucket dredgers, partly where it was too compact for removal by

the former, and partly when time pressed and it was necessary to clear particular areas quickly for commercial purposes. There were also large masses of weeds which clogged and hampered the dredgers and had to be specially dealt with. About 42 per cent. of the whole was removed by the bucket dredgers.

The whole cost of the dredging can be subdivided thus:—

	Per Cent.
Depreciation of plant	20½
Working of the dredgers	69½
Soundings, salaries of temporary staff, etc.	7½
Incidentals	2½
Total	100

The work of the bucket dredgers was too diversified for preparation of any complete subdivision of the whole cost of dredging by them; but the following Table shows the costs at Fremantle of the suction dredgers:—

—	Dredger "Governor."	Dredger "Premier."		
Period in which dredging effected	{ 17/1/1900 to } { 29/6/1898 to } { 3/7/1900 to }	{ 30/3/1900 }	{ 27/6/1899 }	{ 2/7/1901 }
Volume removed in period Cubic yards	221,785	720,377	801,059	
Time dredger delayed in period :—				
For repairs Hours	135	590	408	
By bad weather. . . . "	5	332	410	
By other causes. . . . "	53½	76	241	
Total hours delayed .	193½	998	1,069	
Ratio of delays to whole work- ing-time Per cent.	13	13	14	
Volume dredged per hour of actual time worked Cubic yards	171	111	124	

Items of Expenditure.	Rate per Cubic Yard.		
	Pence.	Pence.	Pence.
On salaries	0·19	0·34	0·24
„ wages of crew	1·23	1·59	1·41
„ coaling	0·08	0·10	0·08
„ watering	0·02	0·04	0·04
„ coals	0·71	0·60	0·64
„ stores	0·06	0·08	0·06
„ miscellaneous	0·16	0·18	0·23
„ repairs and renewals	0·59	1·03	0·94
„ workshops, supervision, etc.	0·14	0·28	0·19
Total	3·18	4·18	3·83

The total cost of the final scheme up to 1903 amounted to £1,458,940, distributed as follows:—

DETAILED COST OF HARBOUR-WORKS.

<i>North Mole:—</i>	£	£
To 3,450 feet—612,700 cubic yards	135,367	
From 3,450 feet to 4,800 feet—394,378 cubic yards	142,747	
<i>South Mole:—</i>		
To 2,040 feet—254,196 cubic yards	70,958	
		349,072
<i>Wharves:—</i>		
On North Mole, 1,000 feet long—51,285 square feet	14,419	
On South Mole, 300 feet long—14,604 square feet	4,992	
Victoria Quay, 5,077 feet long—296,136 square feet	122,686	
North Quay, 1,978 feet long—150,937 square feet	45,173	
Mail-boat jetty, 450 feet long—55,350 square feet	21,728	
		208,996
<i>Reclamation:—</i>		
North side of river—22 acres	22,865	
South side of river—54 acres	27,312	
		50,177
<i>Dredging:—</i>		
Drilling and blasting—1,503,099 cubic yards	238,345	
Dredging—9,637,501 cubic yards	455,298	
		693,643
<i>Contingencies:—</i>		
Breakwind—2,128 lineal feet.	6,302	
Moorings	5,351	
Slipway, with cradle 185 feet long, for dredgers and other vessels up to 650 tons dead weight	21,644	
Office, etc.	4,566	
Land purchase	6,836	
Petty items.	7,383	
		52,082
Total		£1,353,970
In addition expenditure has been incurred on—		
<i>Jetties in open harbour:—</i>		
Main sea jetty	70,960	
Fremantle South	913	
Owens Anchorage, for explosives and stock	3,842	
Woodman's Point, for quarantine station	721	
		76,436
<i>Lighthouses:—</i>		
Rottnest, main.	9,276	
„ eastern	3,114	
„ keepers' etc., quarters	1,619	
Woodman's Point	7,646	
South Mole	2,254	
North Mole, temporary	147	
<i>Cable, mainland to Rottnest Island</i>	<i>4,478</i>	
		28,534
Grand total		£1,458,940

COMMERCIAL RESULTS OF THE FREMANTLE HARBOUR-WORKS.

Recapitulating briefly the progress described in the foregoing pages, the works were commenced in November, 1892, when the first stone was tipped into the north mole; drilling and blasting of the ocean bar was started in July, 1894, reclamation on the south side of the river in August, 1895, and, under great pressure from the commercial community, the south quay was begun in August, 1896. In May, 1897, the works were sufficiently advanced for the first boat, the S.S. "Sultan," an Australian coaster, to make use of the harbour, the entrance-channel being then 180 feet wide and 20 feet deep at low tide. Finally, in August, 1900, the English mail-boat companies set their seal of approval on the works, the first vessel of these lines entering in that month. By the end of 1902 the works were so far advanced that an Act was passed constituting a Trust for management and maintenance, and this body took up its duties in January, 1903.

It is evident from this brief history of events during construction that the harbour was a great commercial necessity. This conclusion is emphasized by the facts that whereas in 1894 the whole tonnage of vessels visiting Fremantle was 337,820 tons (i.e., 406 vessels of an average net register of 832 tons), the corresponding figure in 1902 was 1,322,584 tons (657 vessels of an average of 2,013 tons), and in 1905 it was 1,462,995 tons (797 vessels of an average of 1,835 tons). This trade could not have been accommodated at all at the old outer jetty, on account of both its volume and the large size and deep draught of much of the shipping.

Financially, too, there has been a great saving, for the previous cost of handling, both from the jetty and through lighters, was excessive, whereas the charges in the new harbour are moderate, especially in view of the high wages paid in Western Australia. These charges consist, in the main, of three items: tonnage, berthage, and wharfage. The first, both inwards and outwards, is 3*d.* per ton on the registered tonnage of the ship or, alternatively, on the cargo, if this is less than one-fourth of the net registered tonnage. The berthage is 2*d.* per ton on the cargo, with a minimum of 5*s.* and a maximum of £20; and the wharfage is 2*s.* per ton on the cargo landed.

The net earnings of the harbour were £48,023 in 1903, £50,238 in 1904, and £52,511 in 1905, which on the inwards cargo only amounts to 1*s.* 4*d.* per ton. This may be contrasted with the profits of the old-established harbour of Melbourne, where the net earnings

in 1901 and 1902 averaged 1s. per ton, calculated on a fourfold volume, 2,822,000 tons, of combined inward and outward cargo. Looked at in another way, the financial results are also satisfactory, for the average interest on Western Australian Government loans was 3.41 per cent., and on this basis the annual interest on £1,353,970, the cost of the harbour-works, is £46,170 per annum, or, in round numbers, £2,000 less than the net revenue in 1903. The harbour is therefore one of economical design; moreover, at moderate expense it can be made to accommodate 50 per cent. more shipping than in 1904-5, and there does not seem any doubt that, on increase accordingly of the traffic and endowment of the harbour with storage-facilities, the port-charges could be so reduced as to lower the incidence of the net earnings from the present 1s. 4d. per ton to 8d. per ton, calculated on the inward cargo only.

It has already been pointed out that the late Mr. C. Y. O'Connor, C.M.G., M. Inst. C.E., prepared the scheme originally approved by Parliament. In addition, he controlled all alterations and took an interest in details to the time of his death in March, 1902. Of the Executive or Resident Engineers on these works at various times were Messrs. A. W. D. Bell, M. Inst. C.E., E. H. Carlin, William Leslie, J. A. MacDonald, M. Inst. C.E., G. H. Royce, and, in addition to his regular duties, the Author, who was also similarly concerned from time to time with all the other harbour-works, especially as Mr. O'Connor's successor in the position of Engineer-in-Chief of the State. Mention should also be made of Messrs. C. Good, T. C. Hodgson and J. Thompson, M. Inst. C.E., who dealt at various times as Executive Engineers with the many harbour-works of the State except Fremantle.

The Paper is accompanied by twenty-five sheets of drawings and maps, from which Plate 7 and the Figure in the text have been prepared.

Discussion.

The PRESIDENT, in moving a vote of thanks to the Authors, The President. observed that the members would appreciate very much the fact that important harbour-works in Australia had been brought to the notice of The Institution in three very interesting communications—Papers which showed that the conditions in Australia were quite different from, he might almost say, anywhere else.

Mr. T. A. COGHLAN, Agent-General for New South Wales, remarked Mr. Coghlan. that he was very pleased to have the opportunity of hearing three Papers read on such an important question as the treatment of harbours in Australia. He was hardly in a position to say anything as to the merits of the Papers, and therefore he could not be expected to open a discussion upon them; but he felt that they were meritorious from the point of view of the researches made and the work done by the Authors; and the question whether the ideas propounded in the Papers were good or bad would be discussed by those competent to speak upon such matters. He was very glad to see that the Authors were not unmindful of their obligation to impart to their professional brethren in other parts of the world the results of their investigations and experience in Australia. In doing this they had availed themselves of the opportunities afforded by The Institution, whose work was known wherever civilization extended; and it was a very happy circumstance that engineers in the distant dominions of the Empire were welcomed to lay before The Institution their experience and ideas, and at the same time had the full satisfaction of knowing that they were placing them before a learned and impartial body. He desired to express his appreciation of the kind reception given by the members to the Papers, and he hoped the discussion would be fruitful of ideas and corrective of misapprehensions. Some of the theories put forward—especially those in Mr. Halligan's Paper—were in a sense novel, and he trusted that the discussion would be for the benefit not only of engineers, but also of the various States in whose employment the Authors were, or had been.

Mr. CECIL W. DARLEY observed that in discussing three Papers Mr. Darley. at once it was difficult to know where to begin—and perhaps still more difficult to know where to end. In Mr. Halligan's Paper some

Mr. Darley. new theories were expounded, and while he was glad to see the Paper brought forward and discussed, he could not quite agree with its author's conclusions. Mr. Halligan attributed the movement of the sand on the coast to the littoral current, and pointed out that, by reducing the velocity of the current at the entrance to less than that of the littoral current, the inroad of sand could be stopped. Mr. Darley differed from him on that point. Mr. Halligan instanced Sydney Harbour, Jervis Bay, and Botany Bay, as cases of wide entrances, but it happened that those three entrances had no adjoining sandy beaches; they were entrances through rocky headlands and could not be compared at all with the rivers cited in the Paper. Sydney Harbour had a rocky headland north and south for some distance, and there was no sand-movement there. Thirty-five years' experience on that coast, in close study of the harbour question, had led him to the conclusion there was no travel of sand on the coast due to the littoral current. The movement of sand was local, and due to various local causes, in each case. In some cases there was no doubt that the seas brought in the sand; there was a sandy bottom outside, and the sand was brought into the harbour at right angles and thrown up on the coast; it was not by any means travelling from the north or from the south. Mr. Halligan endeavoured to prove that most of the travel was from north to south. At the Manning River and the Richmond River, the sand-spit encroached on the entrance from the south. On his first visit to the Richmond River, more than 40 years ago, the entrance to the river was nearly 1 mile south of where it was shown on the chart to-day. At that time he saw an incoming steamer struggling to turn at right angles on the bar in order to pass into the river; more recently the entrance had been close up under the north head. The word "Lagoon" on Fig. 7, Plate 5, marked the position of the channel which was seen by Sir John Coode when he visited the harbour. In other words, the entrance had moved more than a mile from the north head to the south. The sand, if it had travelled at all, had come up from the south; it had certainly not come down with the littoral current from the north as the Paper rather would indicate. In all cases on the coast the ebb-current tried to cling to the headland; if the headland was on the south, it would cling to the south; and if on the north, the channel tried to cling to the north. The channel ran up until it met the head at the north of Richmond, and then passed round; but Sir John Coode very properly considered that the channel could not be maintained there, and that there must be a straight run for the river, as straight as possible to the entrance.

The works designed by Sir John Coode, so far as completed, had Mr. Darley. been successful in fixing the entrance, and very fine navigation had been secured, but in the absence of any river-floods for many years some sandbanks had accumulated within the entrance. In this case the sand was partly brought in by the flood-tide, which passed in along the south beach and round the end of the breakwater, and partly, no doubt, it was blown in from the beach over the breakwater. The wind was a fruitful source of sand-trouble at all the river-entrances. Much the same thing occurred in connection with the Manning River. The lagoon shown in Fig. 6, Plate 5, was part of the old entrance, and the sand, in his opinion, was largely brought in at right angles from the sea and distributed along the coast. The Paper properly pointed out that the tidal wave approached the coast at right angles, but in the case of Newcastle it could not be said to do that, as Newcastle entrance was in a deep bight, the coast on the north trending for about 20 miles in a north-easterly direction. The flood-tide made in along this beach, bringing sand in round the breakwater; thus it was the flood-current that brought the sand in, not the littoral current. The sand at the entrance was fed entirely from the north beach.

With regard to the second Paper, the north breakwater was designed by Mr. Darley's predecessor, the late Mr. E. O. Moriarty, who was Engineer-in-Chief in those days, and he himself was there as resident engineer. In making the breakwater the stone had to be landed in punts and run out across the sand-spit at Stockton. At that time the north spit, which was covered with grass, projected towards the channel about 520 feet southward of the line of wall leading to the breakwater; but as soon as the breakwater was advanced beyond the line of sandy beach—thus cutting off, for a time, the inrun of sand—the spit rapidly disappeared under the influence of the ebb-tide, until the wall leading to the breakwater became the water's edge. Gradually, as the breakwater went out, the beach grew out with it, and the sand coming in from the north filled in the bight, following the breakwater, and eventually commenced pouring round the breakwater again with the flood-tide. Before completing the north breakwater, the south breakwater was extended out to sea: it was found that it would not be safe to go farther out without the shelter of the south breakwater, as it was catching the seas and disturbing the shipping in the harbour too much. He designed the outer north breakwater to start farther north and leave a space between as a wave-trap; and so far it had acted efficiently. On the south side a training-wall was run out in order to lift the current up to the north side, and a wave-trap was also

Mr. Darley. made there with a view to protect the harbour. It was clear that the sand had not come in with the littoral current. With regard to making the harbour wider, as Mr. Halligan suggested, it was rather wider now than the current could maintain, and putting the breakwaters farther apart would simply mean having more sand between them. A novel piece of engineering was carried out a few years ago by sinking a series of hulls to form a breakwater, but he was afraid he could not recommend young engineers to try that expedient. In his view it did not hasten the construction in this case by a single day. When large vessels were sunk and reached the bottom, their backs broke and all the sand ran out, and the hulls of the broken ships naturally scattered the stones. In his opinion it was rather a curious expedient, and he did not think it even cheapened the construction. The hulls were now all covered in by the breakwater. He did not agree at all with the view that the sand was brought in by the restricted entrance. On the coast south of Newcastle, under certain conditions, with a gale straight in from the east, he had seen the rocky shore turned into a sandy beach by one gale, the sand being brought at right angles on to the coast. It disappeared again very soon, but the fact showed that the sand was brought in by the sea at right angles and had nothing to do with the littoral current.

Fremantle Harbour was another case of a theory set up in the old days with regard to sand-travel. As the Paper pointed out, Sir John Coode accepted the theory that there was movement of sand along the coast, but of course he was not there long enough to make a study of the subject, and had to accept the views of those in authority at the time. He believed the question of sand-travel and the trouble it caused was first raised by the late Mr. Wardell, who at that time was Director-General of Public Works in Victoria, and who was sent round to report upon it. Mr. O'Connor made a close study of the matter, and knowing that so many engineers before him had accepted the theory of sand-travel, he was very slow and cautious in acting against that theory. Having collected all the evidence that he could get, Mr. O'Connor rightly came to the conclusion that there was nothing to be feared from the sand-travel. Sir John Coode's design was for an open breakwater connecting the mainland with the island harbour, intended to let the sand travel between the harbour and the mainland. As there was no sand-travel to speak of—no permanent sand-travel from the north to south, but only local movement of the sand by the various seas—it had been possible to carry out the works described in the Paper.

Sir WHATELY ELIOT wished to confine his remarks chiefly to Fremantle harbour. The Paper dealing with the Fremantle harbour-works was a very interesting one, as it was a record of the successful completion of work which for years had been described and considered to be almost, if not quite, impossible. In 1874, on his way to New Zealand, he stopped at Albany, and with the natural curiosity of youth began asking questions about the harbour; and the people of Albany informed him that it was the only harbour that would ever be made in Western Australia. The mail-steamers at that time called at Albany only. He was also informed by the Albany people that such a thing as a harbour at Fremantle was quite impossible, and that the Government were about to commence the construction of a railway to connect Perth with Albany, which was to be the port for Perth. Now things were rather different. The railway, 250 miles in length, had been constructed, and so had Fremantle Harbour, and instead of the Perth people going to Albany to embark in the mail-steamers the Albany people travelled to Perth to embark. In the Paper reference was made to Sir John Coode's scheme for a harbour at Fremantle, and that scheme was compared in one respect with a scheme by the same engineer for a harbour at Timaru in New Zealand. The reader might be led to suppose there was some striking similarity in the circumstances of those two places; but that was not the case; the only similarity was in the schemes that were drawn up, the local circumstances being totally different. At Timaru there was a long length of shingle beach, with no sand above low-water mark, and the shingle was constantly moving in one direction. The beach travelled in a continuous line along the coast for about 100 miles, and it was quite evident that it came down by the rivers on to the shore and went to the Banks Peninsula. There was no indentation of the coast, no estuary or river-mouth, but simply the travel of the beach. At Fremantle, on the other hand, there was no shingle but only sand, and there was considerable doubt as to the extent and direction of its movement. In addition to that, there was at Fremantle a large lagoon which offered the tempting prospect of being able to open up a harbour without having to build one entirely outside. In 1908 he had an opportunity of inspecting the Fremantle harbour thoroughly, and he was much struck with the construction of the north mole. He had seen something of rubble breakwaters before, and he naturally asked whether that breakwater had to withstand a very heavy sea. He was assured that it had not, and he did not see how otherwise it could have remained where it was, because, although it was protected on the sea side with very large pieces of granite,

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he concluded, from what he had seen of the effect of seas on rubble mounds in other places, that it could not withstand a very heavy sea-stroke. He was also informed then that there had been practically no silting in the harbour since the breakwater was made and the harbour deepened. He also visited the little harbour of Bunbury, where there was an extensive rubble breakwater similar to the north mole at Fremantle: that breakwater also had evidently not a very heavy wave-stroke, being sheltered considerably by the bold Cape Naturaliste to the south. At Bunbury there was a small estuary, and he was told that the effect of the breakwater was to divert the current coming down in flood-time through that estuary. Before the breakwater was built, the sand used to be deposited just inside the area which was now enclosed; but, since the breakwater had been constructed and carried out to its full length, the current from the estuary seemed to be diverted away from the harbour, and there had been no injurious effect from silting inside. Of course, during the construction of the breakwater the sand—as always happened where it was moving about—followed the breakwater, and a small bank was left behind just inside the end of the breakwater. Mr. Palmer said that it was hoped Nature would remove that small bank afterwards. From experience of the way in which Nature dealt with small troublesome banks, however, it had not been thought advisable to give her an opportunity of doing the work, and the bank had been dredged away.

Mr. Griffith.

Mr. J. P. GRIFFITH remarked that those who were engaged in harbour-engineering knew well the immense difficulty of laying down any hard and fast rules in connection with the subject, and that fact had been peculiarly impressed upon him during a life spent in connection with the harbour-works at Dublin. As many members were aware, Dublin harbour was a classical example of the struggle against encroachments of sand at the entrance to a harbour. He remembered very well discussing, as a young man, several of these problems with the late Sir John Coode, Past-President Inst. C.E., whose acquaintance he made in the Port of Dublin when Sir John was advising the Board of Trade as to the repairs to the base of Poolbeg lighthouse. What impressed him most was the earnest way in which Sir John Coode sought for information on which to base his opinions. Perhaps the chief lesson he had learned from reading the Papers was the importance of the early investigations, on the results of which engineers had to design works involving so much uncertainty as great harbour-works. He had read with particular interest Mr. Palmer's remarks as to Sir John Coode's visit to Australia on behalf of the Government, and his recommendations in connection

with Fremantle. It appeared that it was afterwards discovered that Mr. Griffiths, some of the information laid before Sir John Coode was not correct, and subsequent engineers had had to modify the works accordingly. He simply referred to that matter in order to emphasize the importance of the preliminary work of investigation. In turning to the consideration of the Australian problem one was met at once with the urgent necessity for the works, and the comparative absence of knowledge, before their construction, on the subject of the ocean-currents and littoral currents, as well as of the tidal forces that had to be dealt with. All forbearance should therefore be exercised in criticizing the important works described in the Papers. He thought The Institution was greatly indebted to its members and others scattered abroad for furnishing such information. Nothing was more valuable to the profession than to have these descriptive Papers printed in the Proceedings, for reference in years to come by engineers engaged in similar work. He had discovered during his life that nothing was of greater value than the historical aspect of a port. If Dublin had not had the magnificent surveys made in the year 1820 by the late Mr. Francis Giles, the father of a Past-President of The Institution, he could hardly imagine that the entrance to Dublin harbour would be what it was to-day. About the year 1820 the Government engaged the elder Rennie to advise as to the possibilities of Dublin harbour. He reported that no greater depth could be hoped for on the bar of Dublin than about 8 feet at low water, and he gave up the idea of improving the entrance across the bar, and suggested a ship-canal from a point along the coast adjoining the site of the present Kingstown harbour. Fortunately for Dublin, the harbour-authorities were so impressed with the possibilities of improving the bar that they got Mr. Francis Giles to make a survey of the bay. It was this survey that really determined the future engineering policy of the port, as by means of it the littoral currents, which were the dominating feature controlling the entrance of Dublin port, were discovered. But for this survey, the works carried out by the elder Halpin in conjunction with Mr. Giles could never have been properly designed and executed. The result of these works was that, instead of having 6 feet of water on the bar as in 1820, the construction of the piers alone, without dredging, increased the depth on the bar to 16 feet at low water. That, he thought, was a striking example of the importance of early investigations in connection with harbour-works. He did not propose to criticize in any way the works carried out at Fremantle or at Newcastle; they had been carefully thought out and systematically executed, and had considerably improved

Mr. Griffith. those ports. It should not be forgotten, however, that a great many of the improvements were due to the mechanical appliances which were now at the disposal of engineers. The suction dredger had come to the rescue in places where the bucket dredger was unavailable. The only danger in the future was that engineers might lean too much upon the mechanical side, and on the power of dredging. It was possible to do anything with plenty of money and dredgers, and under those circumstances a port could be maintained even in the face of exceptional difficulties; but he thought that in the first instance engineers should study the physical features they had to deal with, and utilize the forces of Nature as far as possible, supplementing them, if necessary, with the mechanical appliances now available. With regard to Dublin, through the great works constructed in the early part of the nineteenth century, about 10 feet had been gained in depth over the bar. About the year 1880 the bar seemed to come to a standstill, and it fell to his lot to advise as to what could be done. There were various proposals for narrowing the entrance and increasing the scour, and also for increasing the tidal volume thrown on the bar; but with the knowledge already gained he thought himself justified in recommending the Port Board to cut through the bar so as to have 20 feet at low water, and he believed there was sufficient scouring-power behind to maintain the deepened channel. He was glad to say that, as far as the work had gone, his forecast had been fulfilled. The channel had been deepened to 20 feet at low water by dredging, and since dredging was stopped there had been a further increase in depth, due evidently to the scouring-power that produced the first improvement. He could not help thinking that those were lines of procedure which should be kept in view in connection with all such works. Mr. Halligan's Paper was a thoughtful and speculative one, which required very careful consideration. It fell foul of many established ideas of one who, like himself, had been brought up in harbour work, and the only way in which he could approach it was by asking whether the propositions advanced in the Paper applied to the port in which he was particularly interested. Mr. Halligan insisted that the ocean-current was the only sand-transporting influence to be dealt with. That was not true with regard to the Port of Dublin. It might be perfectly true in New South Wales, or on the Australian coast generally, but it did not apply to Dublin Bay. There the encroachment of the sand was unquestionably due to wave-action; the flow of the sand was in the direction of the maximum wave-stroke and of the most injurious wave-action. The tidal currents, ebbing and flowing up and down the channel, produced extremely

little effect in the transportation of sand along the Irish coast, and Mr. Griffiths. all the information at his disposal pointed to the travel of the sand being due to wave-action. He could not help thinking that if Mr. Halligan followed up his investigations he would find that the waves, which were a very important factor according to his own showing along the coast of Australia, were a potent force in connection with the question of sand-movement. When waves rolled over a sandy bottom—he had almost been about to say, no matter at what depth—the pressure of the wave was transmitted to the bottom, and acted more or less as a roller, producing a movement of the sand. His own experience was that even around the deep-water portions of the coast in the Irish Channel there was sand-travel, which he was confident was not due to the main tidal current running up and down the channel. If it were, there would be complete confusion with regard to sand-travel, as there were so many eddy-currents produced by the promontories along the coast. Mr. Halligan also referred to the importance of the width of an entrance being so arranged that the current in and out of the harbour should not be faster than the tidal current, or, in his case, than the ocean-current. That also was contrary to what was met with in Dublin. The great object of the piers of Dublin Harbour—one of which was $3\frac{1}{4}$ miles long, and the other $1\frac{1}{4}$ mile—was to concentrate the last half of the ebb on the bar, which was 1 mile outside the piers. The whole problem of the improvement of the bar channel had depended on the concentration of that ebb from the harbour, at a higher velocity than the adjoining current. At Dublin the ebb-tide was the dominating tide, being faster than the flood-tide, and it was possible to concentrate the ebb on the bar by means of the currents in the bay beyond the piers. The levels of the tidal water north and south of the bar, as it were, bounded the outflow current. He only mentioned that as a reason why he could not accept as a general conclusion Mr. Halligan's reference to the width of entrance as being a dominating factor. He was afraid he had rather unduly dwelt on his own port, but it had been the guiding feature in his mind when reading the Papers. He thought The Institution was greatly indebted to all the Authors, and especially to Mr. Halligan for the fearlessness with which he had dealt with the subject, knowing that his views were not quite in consonance with the views of engineers connected with harbours in this country.

Mr. A. F. FOWLER considered that great importance must be Mr. Fowler. attached to Mr. Halligan's observation that every case of improvement of a river or estuary had to be studied by itself, and any remark

Mr. Fowler, which he himself should make was subject to that postulate. He thought Mr. Halligan was perhaps rather sweeping in saying that too little improvement had been effected for the large amount of money expended. Mr. Fowler must not be taken as detracting from the merit of the work performed by all the engineers who had been connected with the Newcastle entrance, when he said that he was somewhat envious of the almost perfect conditions for improvement that existed in that case. He alluded to the large drainage-area behind, as compared with most English rivers, the comparatively small range of the tide and its low velocity, the bottle-necked formation at the entrance, and the almost entire absence of suspended matter in the water. According to Mr. King, the suspended matter took 1 to 3 days to settle in a tumbler. That was in marked contrast with what obtained in the Humber, where, when he was stationed at Goole under Mr. W. H. Bartholomew, M. Inst. C.E., he frequently noticed, at the confluence of the Humber, the Ouse, and the Trent, between 2 inches and $2\frac{1}{2}$ inches of sediment per tide. In the estuary with which he had been intimately connected for the past 20 years, the Ribble, the amount of suspended matter rolled up by the flood-tide in its earlier stages was 1,120 grains per gallon. One point in Mr. Halligan's Paper was of intense interest to him, namely, the allusion to the fetish of the scouring-power of the ebb-tide. That was a question which, more than any other, had to be considered with regard to the circumstances of a particular case. He might be excused for pointing out that in many estuaries an ebb-tide was only flood-tide water going back again. In order to get increased scouring-power from the ebb-tide, it had to be contracted by means of works of some considerable magnitude, depending upon the range of the tide. Taking the case of the Ribble, where there was a maximum range of tide of 30 feet, the greatest velocity on the ebb-tide was during a short period of half ebb, or a little after, while the greatest velocity on the flood-tide was on the first hour of the flood. It was astonishing, in the case of the Ribble, that throughout the improvement of the Ribble estuary the mental problem had constantly been, how to get from Preston to the sea, ever bearing in mind—almost to the point of obsession—the scouring effect of the ebb-tide. He ventured to think that if the problem had been looked at from the point of view of how to get the sea to Preston, different methods might possibly have been adopted. In order to get increased scour on the ebb-tide it was obviously necessary to contract, and to get the most effectual scour it was necessary to contract up to the height when the velocity of the ebb-tide was at its maximum. In the case of the Ribble

that meant that the upper 4 miles had to be treated by raising Mr. Fowler's training-walls to 20 feet above sea low-water level. Of course it was easy to speak after the event, and he realized now that the works put down to train the ebb so as to make the most of the scour had been of such magnitude that they had reduced the quantity of flood-water, and therefore much scouring-power had been lost upon which dependence had been placed for improvement. The result on the Ribble had been that, whereas in 1850 the sea low-water line approached to within 9 miles of Preston Dock, at the present day it was 2 to $2\frac{1}{2}$ miles lower down. Therefore the natural gradient was decreased from what would be due to any given fall in 9 miles to the same fall in $11\frac{1}{2}$ miles. The estuary above that point, and up to where the river contracted and lost its estuarial nature, had accreted no less than $4\frac{1}{2}$ feet over the whole of the southern portion, and about 7 feet on the northern portion, the latter bearing only the ratio of about 1 to 5 to the southern portion. The result was that 29 million cubic yards of tidal water had been abstracted from the estuary, that was, 29 million cubic yards less of water were ebbing out. It seemed to him, therefore, that in treating estuaries like the Ribble, the Mersey, the Dee, or the Seine, it was a matter for serious consideration whether the idea of training-walls for contraction with a view to encourage the ebb scour should not be dropped, and the fullest use be made of the forces of Nature, as advocated by Mr. Griffith, doing nothing which would go against Nature. When a training-wall was constructed and the flood-tide was found to be attempting to break from the back of the training-wall into the channel, it was obvious that there was something wrong in the position of the wall. Therefore he thought a comparison of the two systems of training and revetting—that was, merely protecting the edges of the channel which Nature formed, so as to prevent it from moving, the principle which was now being adopted by Mr. Lyster on the estuary of the Mersey—was worthy of consideration. During his official connection with the Ribble he adopted the system of putting in low revetting, with a considerable degree of success. Lately the old idea promulgated by the Ribble Commissioners had been reverted to on the Mersey, and a channel had been formed through the sandbank. It was too early yet to express any opinion as to what the result would be, but it would be a very interesting case to watch. On the Mersey the bar was being dredged to admit the flood-tide freely, and the natural channel, the Queen's Channel, was being revetted at Taylor's Bank to keep it in position; and that could be compared with the

Mr. Fowler. Ribble estuary, where double training-walls were leading the ebb-tide through a sandbank straight out to sea. With regard to Fremantle, the question of the travel of sand by littoral drift had been fully discussed by The Institution in 1896 upon a very interesting Paper by Mr. W. H. Wheeler. It would appear that in the case of Fremantle there had been great difficulty in ascertaining the direction of the littoral drift, and the experience gained there would indicate that in cases where it was difficult to determine whether the drift was from north, south, east, or west, it might safely be ignored, as indeed it was at Fremantle. In only two instances had he had occasion to deal with a littoral drift; one was on the Ribble and the other at two places on the Cumberland coast, namely, a small harbour at Harrington and at Workington. In these two places the littoral drift was obvious. There were blast-furnaces on the south side of the entrance to each of these harbours, and both entrances were threatened by the constant travel of slag to the north-west. In the case of Harrington the breakwater had had to be extended about 200 feet seaward, and at Workington a similar extension had been recommended by him, but so far it had not been carried out. The quantity of slag moving along the foreshore was immense, and the travel was particularly striking because, although in each case there were ironworks to the north of the harbour, the north shore was absolutely clear of slag. He had drawn attention to those two cases merely to show that in both the littoral drift was in the direction of the prevailing wind and with the flood-tide. In that respect he thought they bore out Mr. Wheeler's statement and also what Mr. Griffith had said, as to the direction of the strongest wave-action. In the case of the Ribble there was a bank of stone which was driven into the estuary on the north side. The gravel took the line of least resistance, and the angle of incidence of the flood-tide with the foreshore and prevailing wind drove it up into the Ribble. Reverting to the ebb-tide, his experience had led him to resolve never to consider one phase of the tide to the almost total exclusion of the other phase, and he thought Mr. Halligan had done a great service in drawing attention to the fetish of the ebb-tide scour. It was not always a fetish, but sometimes it was a will-o'-the-wisp which led engineers into difficulties from which it was very hard to emerge.

Mr. Davis. Mr. JOSEPH DAVIS remarked that he had the advantage of knowing a little of the circumstances connected with the works referred to in the Papers, and he could say that Mr. Halligan's observations in regard to the New South Wales rivers might be absolutely relied upon. But the conclusions drawn from those observations were

open to criticism, because many of them were scarcely in conformity with the facts. All credit was due to Mr. Halligan for presenting the results of his work and for raising important and difficult questions, even though the views he expressed did not in the main agree with those generally accepted. When the special conditions of the Australian rivers were known, it would be seen how difficult it was to arrive at conclusions, and how doubtful the results of harbour-works would be. Mr. Halligan's remarks as to the effect of the flood-tide appeared to be scarcely in accordance with the circumstances; the facts were, to Mr. Davis's knowledge, as Mr. Darley had stated them, and as had been stated by Mr. Griffith in regard to Dublin. Sand from the ocean was thrown up—possibly to some extent by a littoral current, but principally by the action of the waves, especially during a gale—on to the bar at the entrance to an estuary, and so far as he had been able to gather, none of the sand thus thrown up was carried into the entrance. Silt or sand, or the fine material brought down from the uplands, was carried out to sea in all but very exceptional cases, and any sand found—for instance in the case of the Richmond River entrance—was not derived from the ocean, but was either the result of the works themselves, or was due to drift. Looking at the map of the Richmond River entrance (Fig. 7, Plate 5), it would be seen that there was a spit of sand immediately over the word "South" on the south side of the entrance, and that sand, to his own knowledge, was the result of drift. There was a very long length of exposed sand to the south, and when the wind blew fiercely it carried the sand over the southern breakwater into the channel, and was really the cause of the sand-spit. It would be seen that the channel followed the north training-wall, then shot across to the concave part of the south breakwater and then went out to sea; but, so far as he was aware, the bar itself generally formed, according to the state of the weather, some distance outside the entrance: which showed pretty conclusively that any sand thrown up by the sea remained there until it was removed by the dredger. Another thing that proved to him that that was the case was 'the great demand there always was for suction dredgers immediately after a gale, owing to the quantity of sand thrown up by wave-action. With regard to the effect of the ebb-tide, Mr. King's Paper showed fairly conclusively that the effect of the works carried out at Newcastle had been to move the sand seaward by means of it. There was a definite statement in the Paper that the effect of the ebb was to carry the sand out to sea in the immediate vicinity of the north and south breakwaters, and he was able to confirm that from personal knowledge of the works. With reference to the sinking

Mr. Davis. of the hulks, Mr. Darley would pardon him for differing from the conclusion he had arrived at as to that method of constructing breakwaters. Mr. Davis happened to have been one of the Board who advised that that mode of procedure should be adopted. Mr. Darley would remember that he prepared an estimate for the northern wall and allowed for its construction to a certain depth; but during the construction of the work, after Mr. Darley had left the colony, it was found that the flood-tide and ebb-tide scoured away sand to about 30 to 36 feet below the level at which Mr. Darley expected his wall to be. The result of that was to increase considerably the cost of the wall, as it had to be constructed to a further depth of 30 to 36 feet, and it became a question whether an expenditure two or two-and-a-half times what had been expected was to be incurred. It was then suggested by one of the members of the Board that hulks should be sunk, and the suggestion was thought worthy of being acted upon. It was recognized that when the hulks took their bearings they might possibly break up, but it was thought they might stop the scour that was taking place at the end of the wall; and in the result that expectation was realized. The scour was arrested immediately, and the wall was carried forward rapidly. Possibly some scouring took place on the inner side of the hulks, and the wall on that side would be much deeper than on the outer or sea side. Mr. King described on p. 155 the beneficial immediate and ultimate effects of the sinking of the hulks and punts; and possibly Mr. Darley, when he made his remarks, was not fully aware of the circumstances under which the hulks were sunk, or of the result of their sinking. Mr. Davis had been associated with the harbour-works of New South Wales for some time, but Mr. Darley had been connected with them practically from their infancy up to a year or two ago, and therefore he could speak with most authority. Nevertheless, as far as Mr. Davis's own observations went, he would say that the proposals made by Sir John Coode, modified by Mr. Darley, and carried into effect with such slight alterations as were found desirable from time to time, fully justified the opinion formed by Sir John Coode and the advice he gave. He thought Mr. Halligan was entirely wrong in his idea of leaving the entrance as wide as possible. He could confirm Mr. Darley's statement that in the vicinity of the wide entrances cited there was no sand to cause trouble. The diagram of the Manning River entrance demonstrated fully the peculiar conditions of Australian rivers. The entrances of the rivers often shifted about for long distances, and before the channels were fixed by training-walls the rivers might break through

at any point along the sandy strips which were features of the entrances to rivers in New South Wales. He thought it was highly probable that the sand-spit marked at the Manning River entrance (Fig. 6, Plate 5) between the ocean and the channel itself, immediately above the words "Ballast Wall," had been a moving spit from time immemorial. Respecting the Fremantle harbour-works, he wished to bear his testimony to the good work that had been done. So far as it had gone, the work might be regarded as an artificial basin with an entrance approximately on the site of the Swan River. The Swan River was a very small one. It had simply had to be deepened and the basin made. The work had been costly, and to extend the harbour would cost much more. Two bridges—a railway-bridge and a road-bridge—would have to be removed before any extension could take place. The good work that had been done at Fremantle bore abundant testimony to the sound judgment of the late Mr. O'Connor.

Mr. C. S. MEIK remarked that the Papers of Messrs. King and Mr. Meik. Palmer did not lend themselves much to criticism by those who had not an intimate knowledge of the coasts of Australia, but the Paper by Mr. Halligan was one that was open to very serious criticism by all engineers. It appeared to him that the author of this Paper had started to write with a preconceived idea, and that he had followed that idea all through the Paper. For instance, on p. 136 he said that the point he wished to make was that it was the current and the current only which caused lateral movement of the sand on the littoral. Mr. Halligan had apparently started off with that erroneous assumption—"erroneous" in the sense that it was contrary to the opinions held by engineers accustomed to sea-works—and he followed that assumption throughout and made his facts, and one might almost say his currents, fit in with his theory. For instance, in *Figs. 2* (p. 130) he gave an illustration of a main current setting southward coming against a promontory, being reversed, and travelling northward; that was to say, the current inside was travelling to the north, while the main current on the outside was travelling to the south, the idea being that the current had necessarily to travel north in order to carry the shingle and sand northward and deposit it where it had been deposited at the river-mouth. Of course the most obvious question that would occur to a reader would be: "What becomes of that northward current?" If it travelled north it must go somewhere. It must either pass under the southward current or enter a river-mouth where it lost itself—which was absurd. Mr. Halligan started by saying that each locality

Mr. Meik. must be judged on its own merits: that was quite true, and was exemplified by the case in question. What had really happened was obvious. The sand had been carried northward by the action of the sea due to the prevailing wind in the locality, and the sand had been deposited in the northern part of the bight: in order to account for its being there, Mr. Halligan reversed the current. That led to the main point as to what was the cause of the drift along the shore. As Mr. Griffith and Mr. Fowler had pointed out, there could be no question whatever that the cause of the drift was the action of the waves. It happened sometimes that the action of the current was coincident with it, but, to his mind, there was no question that it was wave-action that caused the sand and shingle to travel along the coast. The motion might be accelerated or retarded by currents, but the fact remained that its chief cause was wave-action. On p. 137 Mr. Halligan said: "Standing on one of these headlands on a bright day, one may often see sand close to the rocks, held in suspension by the water." That, Mr. Meik thought, meant that the contention of the Paper was that the current carried the sand in suspension, which of course with a 2-knot current was out of the question. A 2-knot current might carry silt, or very light material like silt, as could be seen in the Thames, but not sand, and certainly not quartzite sand such as was described by Mr. Halligan. It was true that the littoral currents could cause sand to move, but that sand had first of all to be stirred up by wave-action. The littoral current was not strong enough to raise sand off a beach and carry it along the coast. Mr. Halligan also expressed some extraordinary opinions about harbour-entrances and the scour of the ebb-tide, but that matter had already been dealt with, and Mr. Meik would not labour it. Coming to the question of the width of harbour-entrances, Mr. Halligan had apparently overlooked the fact that harbours were constructed mainly to let ships in and keep the sea out. It stood to reason that, unless the width of the harbour-entrance was restricted, it was impossible to keep the sea out, and therefore the utility of the harbour for vessels was very much diminished. In the conclusions on p. 148, three controlling influences were laid down, and the statements were made that if they were observed they would be sufficient for the designing of any harbour on the coast, and that the problem thus became much simpler than on some other coasts where such favourable conditions did not exist. Mr. Halligan took no notice whatever of the sea, though the sea presumably was a factor to be dealt with on the coast of New South Wales, just as it was on the coast of Great Britain. Therefore Mr.

Meik thought Mr. Halligan's conditions were, to say the least, very Mr. Meik. incomplete. Presumably Mr. Halligan was not an engineer, otherwise he would not have been so rash as to bring such heterodox opinions before The Institution. Like a great many nautical men, he based his conclusions more on what was observed from the deck of a ship than on what had been actually found by carefully surveying the harbours or coasts of a country—an examination civil engineers always had to make when engaged in harbour work.

Dr. J. S. OWENS observed that on p. 130 there were a few Dr. Owens. diagrams illustrating the effect of headlands upon currents, and the first diagram showed a current reversed. He knew of several instances in this country where there were such currents, and the reversed current did go somewhere. Where it went to was, he thought, clear enough: it came back again on the outside. As a matter of fact, some embayed parts of the water behaved somewhat like a pulley with a belt going over it, simply revolving as an eddy. He could call to mind several cases where the drift was determined by the direction of that eddy-current. What he wished chiefly to speak about was the movement of material due to waves and currents, because it appeared to him that the distinction between the behaviour of shingle and of sand had not been drawn by Mr. Halligan or by any of the speakers. His own experience had been chiefly in connection with sea-defence works, and he had found that sand could be moved by a moderately slow current, while shingle could not be moved by so slow a current. He did not know the coast referred to in Mr. Halligan's Paper, but he gathered that shingle was not very plentiful there, and that the conclusions were drawn from observations of the behaviour of sand. He had investigated the movement of sand and shingle, and had determined the actual velocities required for it; and he had discovered a point which he would not go so far as to say explained the behaviour of wide-mouthed harbours, but which certainly struck him as being very peculiar and as having a bearing upon the movement of sand. The ordinary sea-shore sand, composed of quartzite grains perhaps $\frac{1}{30}$ to $\frac{1}{100}$ inch or a little more in diameter, would begin to move along the bottom when the current attained a velocity of about 0·85 foot per second, and it would continue moving for all velocities above that; but the curious point was that there were two distinct stages in the movement of the sand. From 0·85 foot per second up to 2·5 feet per second the sand moved in the form of ripples. Everyone knew the ordinary sea-sand ripple, with a flat slope facing the current and a steep slope away from the current. Between 0·85 foot and 2·5 feet per second, the sand

Dr. Owens moved by being rolled up the flat slope and dropped over the steep crest; it became buried in the trough, and did not stir again until the whole ripple had moved over it. Under those circumstances it moved slowly; in fact, the quantity of sand moved by such a current was trivial. But, immediately the velocity exceeded 2·5 feet per second, the whole behaviour of the sand changed. The ripples were swept away suddenly at that critical velocity, and the sand was raised in suspension—not very high in the water, perhaps an inch or two above the bottom—and was carried away extremely rapidly in a smooth sheet, rushing over the bottom with almost the same velocity as the current. Since the bottom was corrugated by the ripples up to that velocity, shingle was unable to move until the ripples were swept away. The effect was that, although a current of 2·5 feet per second (which he made out to be approximately 1·48 knot per hour) could move a stone between 2 and 3 inches in diameter over a hard bottom, it was unable to move it at all over a rippled bottom, and so the shingle had to stay there until the ripples were swept away. Remembering that 1·48 knot was the critical velocity at which sand ceased to move in ripples and began to move in suspension, it was curious that, in every case which Mr. Halligan had brought forward of a wide-mouthed harbour through which sand was not swept, it would be found that the velocity into the harbour fell below that critical velocity. In Sydney Harbour it was 1 knot, whereas the current flowing outside exceeded the critical velocity. In Botany Bay it was $1\frac{1}{4}$ knot in the harbour—still below the critical figure—and outside it was $1\frac{1}{2}$ knot—exceeding it. The velocity of the tidal current into Jervis Bay was $\frac{1}{2}$ knot, and that of the ocean-current flowing by was $1\frac{1}{2}$ to $1\frac{3}{4}$ knot. That was a remarkable coincidence. He was speaking of currents alone, leaving for the moment the question of the waves. It would almost seem that the explanation that a current must fall below the critical velocity in order to prevent the movement of sand would fit these cases better than the view put forward by Mr. Halligan. He certainly could not see any reason why the relative velocities of the current into the harbour and the current flowing by should affect the amount carried in, with the exception of what was probably a trivial factor due to centrifugal action throwing the grains outward at the bend of the current. A sufficient distinction, it appeared to him, had not been drawn between the behaviour of shingle and the behaviour of sand. In England there was plenty of shingle with the sand, and he knew as a fact that the direction of shingle-drift was determined by the direction of the wave-stroke. As soon as waves came on to the

shore obliquely, the shingle began to move in the direction in which Dr. Owens. it was driven by the waves. But that was not so with sand. Sand went where the current went, and provided it was raised into suspension it would go with any current. When sand had been raised into suspension in water, a hydrometer placed in the water would show a higher specific gravity than if there were no sand in suspension. He had obtained hydrometer readings up to 1.4 by simply shaking sand up in water. Now, the erosive power of a current depended profoundly upon the density of the fluid, and the density of the fluid was increased immediately the sand was raised into suspension. Consequently, it seemed to him that immediately sand was raised into suspension, from whatever cause, the current thereby acquired a much more powerful erosive action.

Mr. A. E. CAREY observed that the points upon which he had Mr. Carey. intended to make a few remarks had been dealt with so exhaustively by previous speakers that he would confine himself within very small limits. As he understood Mr. Halligan's Paper, what its author said was that the great monsoonal current flowing down from the north-west was in conflict with the local tidal currents, and that this condition of things was the cardinal factor in the regime of that particular coast. Assuming that that was correct, as doubtless it was, it would produce a state of affairs which might be compared with the condition of an inland sea or lake. Practically, the normal condition of a coast-line did not appear to be existent at the part of the Australian coast within the scope of Mr. Halligan's Paper. At La Guaira harbour in Venezuela, which Mr. Carey carried out some years ago, a breakwater was run out into about 45 feet of water. There was practically no rise of the tide at all, the maximum range being about 15 inches. Yet on that coast-line there was a very considerable sand-travel. The great feature in the Caribbean Sea was the prevalence of the north-east trade-winds, which blew for 9 or 10 months in the year, stirring up the sandbanks along the coast-line and driving the sand along the shore. The wind waves carried the sand round the breakwater, when constructed, and deposited it to a large extent in the harbour, with the result that steady dredging was necessary to remove it. Mr. Halligan described the ebb-current as the "will-o'-the-wisp of the harbour-engineer." Without that tidal scouring effect, however, nearly all the harbours in the English Channel and in the North Sea would find themselves in very serious difficulties. He had recently carried out harbour-works at Southwold in Suffolk, where there was a small tidal river flowing down, with a tidal compartment of only 7 miles and a catchment-

Mr. Carey. area of only about 70 square miles. When the works were commenced the entrance was practically blocked; after a prevalence of southerly winds there was only 2 or 3 feet of water between the pier-heads. The piers were projected into the sea, leaving an entrance of 123 feet width. Off the harbour, at a distance of about 50 yards, there was a steep shoal or bar. The construction of the piers and the guiding of the ebb-tide effected by them had resulted in a depth of water of 16 feet at the entrance to the harbour at low water, and in the disappearance of the bar, and the internal natural scour had deepened the harbour to a very considerable extent. It had been deepened partly by dredging, but largely by the action of natural scour. He had recently devised a system¹ which might perhaps be of service in such cases—a mechanical eroder intended to be utilized where there were sand-banks and shallows, which, if thrown into a state of suspension, could be carried away beyond the zone of the harbour. He had made a number of experiments with the apparatus on the Dutch coast, and he believed it would be of great service in many cases, especially in dealing with silt and with material readily placed in suspension. Mr. Palmer's Paper was an admirable one, especially as the Author gave such full and complete data with regard to the cost of various sections of the work. If he would kindly furnish data as to the life of jarrah timber in the sea, they would be valuable. Mr. Carey had recently used jarrah for work in the English Channel and in the estuary of the Thames, but there was very little available information as to how long jarrah timber was likely to resist the attacks of the teredo in British waters.

Mr. Hudleston. Mr. F. HUDLESTON remarked that one small paragraph in Mr. Halligan's Paper showed that he did not rely altogether on the current alone to pick up the sand. On p. 139 he said: "When the sand, put into suspension by the waves, and slowly moving with the current along the beach, at length reaches an inlet, it is carried into the estuary by the flood-tide." That was rather interesting, as it did not quite agree with some of the remarks that had been made in the discussion.

Mr. Jordan. Mr. WM. LEIGHTON JORDAN observed that the passage referred to by Mr. Hudleston put a different complexion on the matter from that which had been given to it by several of the speakers. He thought everybody would agree that waves raised the sand, and the sand, being raised, went away with the current. The idea of the main features of oceanic circulation being due to the

¹ British patent No. 26,442 of 1910.

winds was now not universally accepted, the newer view regarding Mr. Jordan. it as erroneous; and, as a matter of fact, if the Australian east-coast current came from the position indicated in Fig. 1, Plate 5, it would not be so distinctly a warm current. Admiral Wilkes,¹ of the United States Navy, had first declared that stream to come from the east of New Caledonia, and Commodore Maury² was of the same opinion. But Wilkes's own temperature-chart showed the origin given in Mr. Halligan's Figure to be a region of cold water flowing northward from the east of New Zealand; and the "Challenger" records had since shown that there existed a great mass of water, flowing from the equator west of the longitude of New Caledonia, decidedly warmer than the water east of that longitude. Admiral Lutké,³ of the Russian Navy, had shown that in the south-east trade-wind region of the Pacific Ocean, there was no current flowing with the wind. Mr. A. G. Findlay⁴ recorded that a counter-current in the North Pacific ran steadily right against the trade-wind for two weeks. Admiral Wilkes⁵ had further shown that in the region of Saint Helena, in the heart of the south-east trade-winds of the South Atlantic, little or no current was ever experienced. Finally, Major Rennell⁶ had previously pointed out that experienced ship-captains knew that in the North Atlantic there was a broad stream which helped them when beating northward against the north-east trade-wind. The records by Wilkes, Maury and Lutké, merely showed the impotence of the trade-winds in the South Atlantic, the South Pacific and the North Pacific, to be the same as described by Rennell in the North Atlantic. The "Roaring

¹ C. Wilkes, "Narrative of the United States Exploring Expedition," vol. v, p. 473. London, 1845.

² H. F. Maury, "The Physical Geography of the Sea" (General Chart of Currents). London, 1857.

³ F. Lutké, "Voyage autour du Monde." Partie Nautique, p. 186. Saint Pétersbourg, 1836. "En latitude de 26° nous reçûmes un vent de S.E., qui passa insensiblement à l'état de véritable vent alisé, et qui même quelquefois souffla fraîchement; mais tout cela ne produisit point de courant; . . ."

⁴ A. G. Findlay, "A Directory for the Navigation of the North Pacific Ocean," 3rd ed., p. 1200. London, 1886. "On the Equator, in long. 175° E., a current of about 2 or 3 knots an hour ran to the eastward for 14 or 15 days, although the wind was then fresh from the eastward; . . ." Findlay, on the same page, records that Admiral Wilkes traced the North Pacific counter-current from long. 170° E. to long. 138° W. Maury, in paragraph 401 of the work above quoted, says that the currents of the calm belt of the Pacific are very strong, and are generally found setting to the west.

⁵ Vol. v, p. 469 of the work cited above.

⁶ J. Rennell, "An Investigation of the Currents of the Atlantic Ocean," p. 115. London, 1832.

Mr. Jordan. Forties," alluded to by Mr. Halligan, created long sweeping seas in the South Atlantic, where the eastward current caused by the earth's rotation went with them; but when they met the Agulhas current coming from the east the character of the great waves changed. The current chafed against the wind, forming short, dangerous seas, but it held to its course unchecked: showing that the "Roaring Forties" had as little to do with the creation of the eastward currents of the temperate zones as the trade-winds had with the westward currents of the equatorial regions. Mr. Halligan's Fig. 1 gave correctly the main features of the Australian coastal currents, but it was certainly incorrect as regarded the region from which the warm water on the east coast was derived.

Mr. Pou'den. Mr. G. E. L. POULDEN considered that there were many points raised in the Papers which younger members of The Institution would like to see thrashed out by their seniors. His experience had been gained chiefly on the west coast of South America, where he had had an opportunity of studying the matter under Mr. Adam Scott, M. Inst. C.E., consulting engineer to the Chilian Government. Most of the diagrams shown would apply to that coast to some extent if the cardinal points were reversed; and under those conditions the Richmond River would resemble one river-mouth which he had studied and helped to survey, the mouth of a comparatively insignificant river, which it was hoped to convert into an important harbour. It seemed to him that in all such cases due regard should be had to the geology of the locality and the age of the river, and the fullest investigation should be made as to what the river had been doing in geological times, and what it was trying to do now. Care should be taken to ascertain whether it was endeavouring to assist the engineer or not, and due consideration should be given to the question whether the harbour was to be designed for coastal traffic or for over-sea traffic. In the case to which he referred the harbour was only intended to admit at first vessels of light draught, the river not containing sufficient water to scour to any large extent; and it was concluded that the sea was having a marked though not complete effect in damming up the mouth. At one time the water on the bar had not exceeded 5 feet—if there had been as much as that. He ventured to think that the scour of a river had a good deal to do with the maintenance of the depth of water at its mouth, and that conclusion was exemplified by Fig. 4, Plate 6. It was stated that a great freshet in the Newcastle river occurred in 1893, and in 1896 the result of that freshet was still visible, the 5- or 6-fathom line showing that the effect lasted at least throughout 3 years, because, as he gathered, no works were

carried out in the meantime to the north, and the south break-
water was extended only slowly. It would be interesting to know
how the soundings had been taken. Generally his view was that
the position by sextant should be plotted to about a metre, or less,
of the position at which it was actually taken, no matter at what
distance from the land, always supposing that the leading marks
were good. Section-lines were of course more reliable, and it would
be interesting to know whether the soundings on which the contours
were plotted had been taken by sextant or by section-lines from
leading marks. With reference to the cost of dredging, he thought
it would interest engineers to know whether both deck and engine-
room stores were included in the return headed "Stores," and also
whether the time of the dredger going to her place of deposit and
returning was included.

Mr. HALLIGAN, in reply, remarked that he was afraid a good deal
of the adverse criticism of his Paper arose from misapprehension of
the facts put forward in it. Mr. Darley had said that because the
entrances to Sydney Harbour, Jarvis Bay, and Botany Bay had
rocky headlands on both sides, and no adjoining sand beaches, they
could not be compared with the river-entrances cited in the Paper.
Careful perusal of the Paper would show that they had not been
compared, but that they had been referred to as illustrating the fact
that in no case on the coast of New South Wales did a broad entrance
exist having shallow water, while in no case did a narrow entrance
exist having deep water. The Paper had been written mainly in
the hope of eliciting some explanation of these facts other than that
offered by its author. To him it appeared that the north and south
heads of Sydney Harbour represent the north and south break-
waters which should be built at the entrance to a sandy estuary.
The fact of their extending for some distance on either side was
immaterial, for the sand would travel as readily past a headland
a mile long as past a point a few yards long. Mr. Darley had said
that there was no movement of sand at Sydney Harbour entrance,
but he had not explained how the sand in the eastern channel
had accumulated, as mentioned on p. 141. Mr. Halligan was of
opinion that the Paper read before The Institution in March, 1896,
by Mr. W. H. Wheeler,¹ and the discussion which it provoked, com-
pletely disposed of the idea of sand being brought in from the depths
of the sea: at all events, any such statement would require to be

¹ "Littoral Drift: in its Relation to the Outfalls of Rivers and to the Con-
struction and Maintenance of Harbours on Sandy Coasts." Minutes of Proceedings
Inst. C.E., vol. cxxv, p. 2.

Mr. Halligan. backed by very strong proof, while Mr. Darley made the statement without one observation to support it. Mr. Halligan did not expect all engineers to agree with his conclusions, but he thought that before his observations were disputed, other facts and figures should be produced to disprove them. On the coast of New South Wales so many hundred observations and measurements had been made during the last 20 years, which proved beyond all doubt that the beach-sand was travelling from north to south, except in the few cases cited on p. 135, that it was not proposed to discuss the question now, unless some definite records were produced to show the contrary. In referring to the building of the Newcastle north breakwater, Mr. Darley had described how "the sand coming in from the north filled in the bight, following the breakwater, and eventually commenced pouring round the breakwater again with the flood-tide." This was exactly what Mr. Halligan contended must necessarily happen when the entrance was made so narrow as to create a high velocity for the incoming tide. Was it not quite evident that, had the entrance been sufficiently wide, the sand now drawn in at each flood-tide, would have been carried past in the littoral current? Mr. Darley said: "With regard to making the harbour wider, as Mr. Halligan suggested, it was rather wider now than the current could maintain . . ." If this harbour-entrance had to be maintained by the tidal current, then Mr. Halligan at once admitted that wide entrances were a mistake; he was of opinion, as stated in his Paper, that the less tidal current there was the less sand would be brought into the harbour to cause trouble to the engineer. The waves were undoubtedly a very potent factor in connection with the question of sand-movement on coasts where no permanent ocean-current existed, or where the currents were tidal; for then the strongest wind, which raised the heaviest sea, must temporarily cause the strongest current, and so move the largest quantity of sand. It was not, however, in Mr. Halligan's opinion, correct to say that the waves caused the sand-movement, but rather that the wind which caused the waves also created a current which moved the sand when stirred up by the waves. The propositions advanced in the Paper did not necessarily apply to the Port of Dublin, in which Mr. Griffith was interested, or to any port where the same conditions did not obtain as on the coast of New South Wales. In Dublin Bay the ebb-tide was, as Mr. Griffith had stated, faster than the flood-tide, and therefore the resultant movement of the sand at the entrance must be in the ebb-tide's direction. This was pointed out on p. 147, and Mr. Halligan might perhaps be pardoned for calling attention again to the fact that his statements as to

width of entrance were not to be taken as general conclusions, Mr. Halligan. but only as applying to the harbours on the coast of New South Wales. Mr. Davis had spoken of the bars at river-entrances being formed of sand thrown up by the sea during a gale, but Mr. Halligan contended that there was absolutely no warrant for such an expression. All writers on the subject, so far as he was aware, agreed that the function of the storm-wave was to tear the beach and the shoals down and carry the sand seaward, and that such sand was washed up again during normal weather. On the coast of New South Wales sand was sometimes heaped up at the northern end of beaches, and to a less extent at the southern end during southerly gales; but, as stated in the Paper, this was due to the wind retarding the littoral current, whereby less sand was conveyed past the entrances than during normal weather. Without going into the details of wave-action to prove what must be regarded as an accepted fact, it might be pertinent to ask why the sand did not heap up, by the action of the waves, at Sydney Harbour entrance, Hawkesbury entrance, Port Stephens, Jarvis Bay, etc. The sea-bottom at each place was clean white sand similar in every respect to the beach-sand at Newcastle, the storm-waves were as large, the wind was as fierce, and the entrances had the same degree of exposure. It was impossible to believe that the storm-wave had a selective action, as must be admitted if it were desired to regard the words "thrown up by the sea" as anything but a phrase. Mr. Meik had paid him the compliment of saying that he had started to write his Paper with a preconceived idea, and had followed that idea all through the Paper. Mr. Halligan admitted this: authors generally had a preconceived idea before starting to write. The measurements of the speed, direction, volume, and salinity of the ocean-currents had certainly been made from the deck of a ship or from a boat, but the direction and volume of the sand-movement had been ascertained by observations and measurements ashore, and from careful surveys of nearly every river and harbour on the coast during a period of about 30 years. Mr. Halligan did not claim to be an engineer; and as to bringing heterodox opinions before The Institution, he was of opinion that all progress must necessarily be over the dead bodies of what were once orthodox opinions, and he had submitted his Paper to The Institution, knowing that it would be judged on its merits alone. He did not claim originality for the broad-entrance theory, except in so far as it referred to the harbours of New South Wales, where the conditions were suitable for its application. Mr. W. H. Wheeler had laid

Mr. Halligan. down, in 1896, as a general principle for the guidance of engineers designing harbours on coasts subject to littoral drift,¹ "that the entrance to a harbour should be sufficiently large to prevent any strong set into it on the rising tide, and to allow of its being filled with a smaller velocity of current into it than the flood-tide has in front of the entrance." If for flood-tide ocean-current were read, this principle applied perfectly to the coast of New South Wales.

Mr. King. Mr. C. W. KING desired to thank the members for their courteous reception of his Paper. In reply to the discussion upon it, referring to Mr. Darley's remarks as to widening the Newcastle entrance, the proposal of the present Chief Engineer was to extend both breakwaters on existing lines to such positions that the north wall would arrest the sand-travel during flood-tide from the north, so that it could be handled easily by occasional dredgings; and the south wall extension would still further protect the entrance-channel from the heavy south-easterly seas, and prevent any additional range in the harbour, which would probably occur if the north wall were extended without an addition to the south. This proposal, however, would not widen the entrance appreciably. The purpose of the comparative contoured plans up to that of 1907 was to show the beneficial effect of the ebb-tide current in maintaining the depth of channel at the existing width. In regard to the sinking of the hulks, the saving of cost was shown by the difference in the quantities of stone per lineal foot required to build the breakwater. For the 400 feet constructed immediately preceding the sinking of the hulks the quantity of stone per lineal foot (of breakwater) totalled 208 tons; whereas, over the length blanketed by hulks and iron punts, the quantity was reduced to 135 tons per lineal foot. He would point out, however, that the opportunity for this class of construction was unique, inasmuch as the wrecks of the "Adolph" and "Regent Murray," as well as the "Lindus," assisted considerably in the blanketing process. He did not suggest that the inflow of sand was due to the restricted entrance, but attributed it solely to the flood-tide current round the end of the north wall, particularly in north-easterly and stormy weather. With regard to Mr. Griffith's remark that many of the improvements were due to mechanical appliances such as suction dredgers, Mr. King might point out that the contours of 1907 showed a vast improvement over those of 1905, due very largely to the scouring effects of the breakwaters, the dredging during this period being equalized by the inflow during each flood-tide; and it was anticipated that the effect of the proposed

¹ Minutes of Proceedings Inst. C.E., vol. cxxv, p. 28.

extension of the north wall would be to reduce still further the Mr. King. necessity for dredging operations. In regard to the system, mentioned by Mr. Poulden, of taking the soundings from which the contour-plans were prepared, the practice of running parallel lines across the channel had been adopted, every fourth sounding being located by sextant angle off a section-line on to fixed stations at convenient spots, the intermediate soundings being interpolated with sufficient accuracy for practical purposes by keeping a uniform speed on the sounding-boat. The degree of accuracy aimed at was the plotting of the figure of the corrected sounding, covering, on a plan of 400 feet to 1 inch, the position where the sounding was taken. On a sandy bottom this practice gave the necessary degree of accuracy, but on a position covered by uneven boulders or rock formation, lines were taken longitudinally with the current, the lead feeling the bottom as the boat was allowed to drift slowly. When crossing each 100 feet section the position was taken as mentioned above, and the intermediate depth again interpolated. Calm weather was selected for the latter operation, and in important places the results were checked by repeated driftings. The accuracy of this system had recently been proved by a sweep 40 feet long, fixed horizontally at a certain depth, previously ascertained in the ordinary way as the minimum depth of the channel, and carried by a steam lighter over the portion under examination. Any depth less than that so recorded on the sweep was readily observed from the deck of the lighter. The design and operation of this sweep was under the supervision of Mr. Percy Allan, M. Inst. C.E., whose results therewith as to depth of channel concurred with those of the Author. In regard to Mr. Halligan's remarks on this port, and the statement that the bar shoaled from $22\frac{1}{2}$ to 18 feet, this effect was shown on the 1905 plan where the 3-fathom contour touched the line of leading fairway towers. There still remained, however, the 22-foot navigable channel to the south of this line. The 38,000 tons of sand removed in 6 weeks had been dredged not from the bar but from the extension of the line of the north breakwater. This had been done as an experiment, in order to ascertain the possibility of arresting the inflow of sand from the Stockton Bight by dredging alone, without the assistance of an extension of the north wall.

Mr. PALMER, in thanking Mr. Coghlan for his kindly remarks, Mr. Palmer. observed that he could not claim to be an Australian engineer by birth, education, training, or experience. He was now in practice in England, and his Australian experience, however valuable, had covered a comparatively short period. He could not see how the remarks in his Paper as to dredging could lead it to be supposed,

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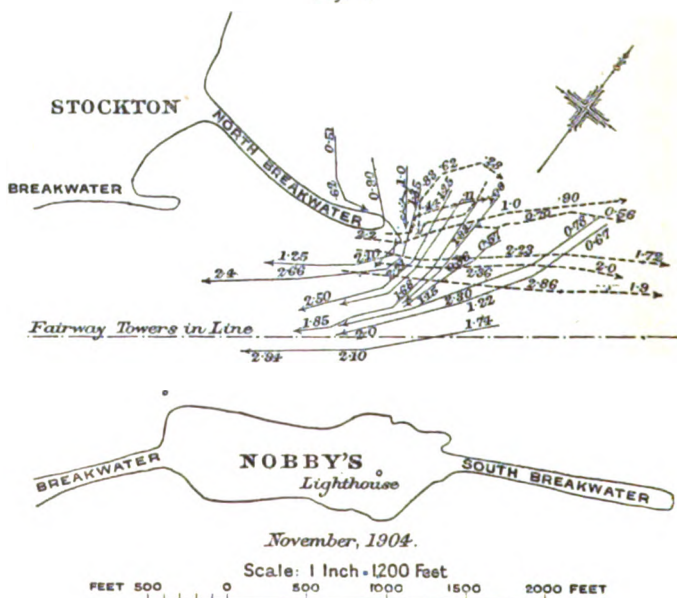
Mr. Palmer. as feared by Sir Whately Eliot, that there was similarity in the physical conditions of Timaru and Fremantle. Sand-travel might exist, and might even be of equal amount, without any such similarity. He was obliged to Sir Whately for pointing out that it had been necessary to dredge away the sandbank which had accumulated at the end of the Bunbury mole. The accumulation took place subsequent to construction of the work, and Mr. Palmer had put on record, before leaving Australia, his opinion that dredging would be necessary. The dictum that Nature would do the work was a subsequent and local public enunciation. The answer to Mr. Poulden's queries was that the cost of all stores was included in the dredging return, as also was the time spent by the dredger in going to and from the place of deposit. In reply to Mr. Carey, it was to be regretted that no information had been published regarding the life of jarrah in British waters. Mr. Palmer considered that this timber could not claim immunity from attacks by the teredo, though it certainly resisted attack better than softer woods. The Government of New South Wales had issued a publication giving their experience with the timber, and Mr. Palmer believed that the results of tests made in Holland were published a few years back, though he could not find out where they had appeared. In the vicinity of Fremantle jarrah was practically not attacked by the teredo.

Correspondence.

Mr. PERCY ALLAN considered that the proposition put forward by Mr. Allan. Mr. Halligan, to design harbours on the common basis of 1 foot of entrance-width to 4 acres of tidal compartment, would appeal to few engineers; and as bearing on the question of the late Sir John Coode's opinion in regard to the benefit of contracted as against wide entrances, the case of Lake Macquarie, situated about 16 miles to the south of Newcastle, might be cited. This harbour had an area of 27,000 acres, and the width between the breakwaters was about 1,400 feet. There was no large volume of upland water after heavy rains, the large lake acting as a regulating reservoir and allowing of the gradual discharge of flood-water through the entrance. The works were designed in the early days of the colony, and after about £98,000 had been spent without any resulting improvement, it was decided to obtain a report from Sir John Coode: he reported in 1887 that to make an effective entrance between the ocean and the lake it would be necessary to reduce the width at the ends of the breakwaters to 200 feet, at a cost of £63,000. This report led to the stoppage of the works. Mr. Halligan stated that at Newcastle the tidal compartment was 10,288 acres, requiring an entrance at least 2,570 feet wide, as against the existing width of 1,200 feet, yet Mr. King gave the tidal compartment as only 6,500 acres, which would reduce this width to 1,625 feet; this was an entrance even narrower than obtained under natural conditions when small vessels only could enter the port, the chart at that time showing depths of but 10 feet and 14 feet respectively in the channels, and an entrance disfigured by wrecks. The later charts accompanying Mr. King's Paper also showed the many subsequent wrecks at the entrance, due, in Mr. Allan's opinion, to the then wide entrance and uncontrolled cross ebb-currents setting to the north and forcing vessels on to the much-dreaded "Oyster Bank." Again, the normal speed and direction of the current off the entrance was given by Mr. Halligan as $1\frac{1}{2}$ knot per hour to the south, but Mr. Allan would point out that the port of Newcastle was embayed by Morna Point to the north and by Red Head to the south, and the prevailing southward ocean-current was not met with at the entrance. This was clearly shown by the current observations on the accompanying charts taken by Mr. Surveyor Kenny in 1906 (*Fig. 1*) and by Mr. King himself in 1909 (*Fig. 2*),

Mr. Allan. and both charts also showed the ebb slightly to exceed the flood-tide. Again, the top leading tower of the port being situated in the grounds attached to Mr. Allan's residence, he had had exceptional opportunities during the past 3 years of observing continually the set of the current off the entrance, as clearly indicated by the soiled river-water. On the ebb-tide this ran straight out for more than a mile, gradually fanning out with a northerly set towards the Stockton beach, the soiled water, after continuous north-easterly weather, being occasionally discernible for a mile to the south. It was also a matter of common knowledge that the debris in flood-time

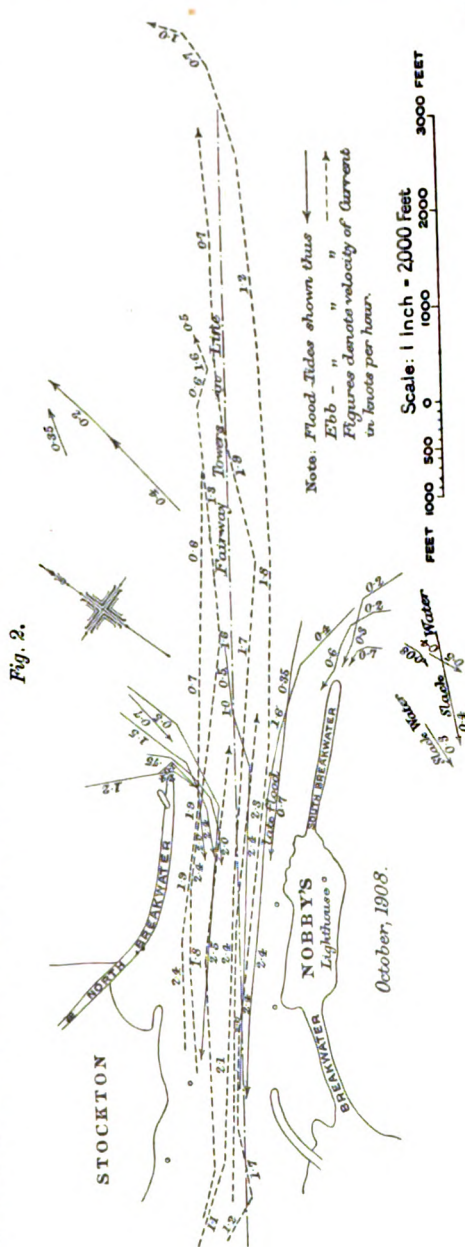
Fig. 1.



was often carried as far as Morna Point, 18 miles to the north, whilst floating objects carried out of the port in ordinary weather were usually thrown up on Stockton beach; so that in the absence at the entrance of a $1\frac{1}{4}$ -knot ocean-current setting normally southward, Mr. Halligan's proposed method of determining the required width of entrance would not appear to be applicable to the port of Newcastle. Mr. Allan considered that current-observations made in a gale of 60 miles per hour were of no practical value, even though recorded to 0.3 knot, especially as the velocity of 60 miles per hour—possibly a local gust—differed from the official registration at

Mr. Allan.

Signal Hill, Newcastle, of 44 miles per hour as the maximum wind-velocity for the month of February, and 51 miles for the year 1908. While it was true that sand was brought into the Newcastle entrance with the flood-tide, Mr. Allan considered this to be due to the north breakwater having been stopped in comparatively shallow water close to the Oyster Bank; the waves breaking on this bank stirred up the sand, which was then carried into the entrance and deposited near the old end of the breakwater. With the 420 feet extension of the north breakwater now under construction, the end would be in deeper water and be so far seaward of the Oyster Bank supply of sand as materially to reduce the amount of dredging at the entrance. Mr. Halligan had, from a theoretical point of view, so questioned the design of the Newcastle entrance as to warrant some reference being made to the practical result obtained. Mr. C. W. Darley,



Mr. Allan. M. Inst. C.E., who designed the works, said, in a letter written to Mr. Allan, that on the day he arrived in Newcastle (12th June, 1867) he went out on the bar and took soundings with the Harbour-Master at that time, Captain Allen, and found only 14 feet. He also said that there was then only berthing for one oversea vessel, and one mooring-buoy for an oversea ship in the Horseshoe—a contrast to March, 1910, when more than a hundred deep-sea vessels were, at the termination of the strike, awaiting coal-cargoes. With the exception of about 185 feet of the north breakwater, now in course of construction, the works as designed by Mr. Darley had been completed, and it was doubtful whether more successful engineering and commercial results had been obtained in the construction of any artificial harbour-entrance. Since the completion of the first 3,000 feet of the north breakwater, early in 1907, many heavy gales had been experienced, yet not a single wreck had been recorded, although between then and the present date no less than 11,100 vessels, with a total registered tonnage of upwards of 20 million tons, had passed safely in and out; in fact, the coal-shipments alone for the record year of 1908 totalled more than 6 million tons. These figures did not include the local tugs, in many cases with barges in tow, which formed a not inconsiderable portion of the shipping business of the port. Vessels with a draft of more than 25 feet had left the port on a neap-tide, and many drawing 25 feet 11 inches had left at high water, the S.S. "Drachenfels" having cleared in March, 1911, carrying 10,588 tons of coal with a draft of 26 feet 10 inches. For many years the potentialities of Newcastle as a deep-sea port had been considered to be limited by the occurrence of solid rock at the entrance, which could be removed only at heavy expense. Upon taking up the position of District Engineer at Newcastle in April, 1908, Mr. Allan was requested by the then Chief Engineer, Mr. L. A. B. Wade, M. Inst. C.E., to have an examination made as to whether the rock was of such a jointy character as would permit of explosives being used without the cost of drilling; and, shortly after, a punt was moored at a point in line with the fairway towers where only a depth of 21 feet 9 inches obtained. The diver, after sending up a number of boulders, reported that he could work a bar down 4 feet without touching solid rock. A couple of clam-shell dredgers were then set to work, with the result that a test hole, excavated to a depth of 32 feet 6 inches, was found to be still in shingle. Since then upwards of 18,000 tons of chert boulders had been removed from the entrance-channel, and the depth of the port as advertised to the world had been increased from 20 feet to 23 feet at low water

of spring-tides. The first additional 2 feet was mainly due to the **Mr. Allan.** widening of the channel by the sand-pump dredger "Jupiter," the gutter on the Green Beacon line carrying 22 feet 6 inches not being recognized by the Navigation Department in signalling the depth of water on the bar, which was based on the 20 feet then obtaining in line with the leading fairway towers. Although the clam dredgers were doing their work well, yet when the calm westerly winter weather set in, the present Chief Engineer, Mr. E. M. de Burgh, M. Inst. C.E., purposed employing a ladder dredger, which it was anticipated would have no difficulty in picking up both the large boulders and the underlying shingle. The Navigation Department being responsible for giving the draught to masters of vessels leaving the port, it was necessary—before advertising any increased depth—to determine that no projecting boulders had been left in the channel, and for this purpose a sweep had been designed by Mr. Allan which consisted of a steel pipe 10 inches in diameter and 40 feet long, rigidly connected to two 8-inch by 8-inch Oregon-pine vertical runners, spaced 25 feet apart. The sweep was placed under a steam-lighter, the vertical runners coming up on either side and passing through slots in a heavy timber frame rigidly bolted to the deck; steel guy-ropes were made fast to the horizontal pipe, passing fore and aft to keep the structure in a vertical position. The sweep was then lowered to the required depth to be tested, the depths being marked off on the vertical runners and adjusted to low water of spring-tides as the tide rose or fell, the tide-gauge readings being signalled to the lighter for every inch of variation. The width of channel was buoyed off and the lighter was steamed up and down, taking a strip of 40 feet at a time, until the whole channel had been gone over several times without touching. In using a sweep such as this, the results obtained, although on the safe side and quite satisfactory and convincing to the Navigation Department, were usually against the engineer's work, the pitching of a short vessel even in a slight sea-way resulting in a less depth being recorded than on a calm day.

Mr. WM. REID BELL considered that the thanks of engineers in **Mr. Bell.** Australia were due to Mr. Palmer for his useful record of works executed at Fremantle, a port whose importance in the Commonwealth was second only to that of Sydney. The old idea of detached sheltering breakwaters on sandy coasts seemed to have died very hard. It was difficult to understand how anyone could imagine that, with a drift alongshore induced wholly or in part by wave-action, a solid obstruction to the wave-action would permit the passage of the drift-sand, and the delivery thereof at the further

Mr. Bell. end of the protecting work, to be carried on its previous course just as if no break had occurred in the propelling force. Like many other engineers, Mr. Bell, in his day, had thought out all sorts of contrivances to train and concentrate the shore currents, say, in the fashion of a Venturi meter, with the object of forcing the drift past the breakwaters and delivering it unsettled at the other end; but, however far one went, one was brought up at last against the fact that to every shelter there must be a lee somewhere, and there must the current fail and the drifting of sand sink to the bottom. The approaches to Fremantle seemed to be well lighted, but he questioned the economy of employing a first-order light in such a situation as a leading light, particularly in a sandy channel where variations in the banks and shoals might occur from time to time. Such a variation had actually occurred at Fremantle, as a large extension of the shoal east of Middle Bank was located by H.M. ships in 1907. This shoal, carrying about $4\frac{1}{2}$ fathoms, invaded the white sector of Woodman's Point light, and vessels coming in had to navigate on the border between the white and green sectors. This was not a satisfactory result for the expenditure of £7,600 plus the annual cost of a first-order establishment, although, of course, it must not be lost sight of, that, in the few years since the Woodman's Point light was established, great strides had been made in the reliability of untended lights. The instance, however, served to show the necessity for providing flexibility in systems of channel-lighting. Two or three years ago there was some desire on the part of mail-boats for a landfall light on Leschenault Point, or Wreck Point, some distance to the north of Fremantle. The currents in the offing set towards the shore for the greater part of the year, and it was contended that in the case of a high-speed ship running to time and making the landfall from the northwest, there was nothing to guide the ship to Rottnest in the event of the current setting her in towards the coast. This coast was devoid of prominent landmarks, and as it was fringed with shoals for a distance of 8 or 10 miles, beyond which the soundings deepened somewhat rapidly, the effective and economical lighting of it was a matter of difficulty. The Paper enabled an interesting comparison to be made between river-ports and harbours on the open coast. It was impossible, of course, for an outsider to range before him all the circumstances which had determined a question of harbour accommodation; but, on the other hand, the invisibility, so to speak, of local conditions to an engineer at a distance enabled him to view the matter in a different if not a clearer perspective—having regard to what ought to be done, as distinguished from

the political or local possibilities open to the man in command. Mr. Bell. Mr. Bell therefore advanced his opinions, not so much as criticism of a good work, but as considerations to be taken account of in coming to a decision upon schemes of modern harbour accommodation. Modern appliances rendered possible, and modern requirements demanded, departures from the harbour-engineering of Smeaton, Telford, Rennie, and Watt—for Watt was a harbour-engineer. Not that the principles laid down by these masters were at fault; but the accommodation required to-day, and the means of utilizing or combating the forces of Nature had grown so that present-day harbours were problems of a different order. More particularly did these remarks apply to the questions of deepening and maintaining harbours by natural or induced scour, and to the utilization of natural as opposed to the creation of artificial harbours. The questions that had faced Sir John Coode, and later, that accomplished engineer, Mr. C. Y. O'Connor, might be taken to be sand-drift on the coast, and the utilization of the Swan River. Sir John's choice was right, inasmuch as he rejected the river and chose the foreshore in front of Fremantle, where the space for expansion was unlimited; but his scheme of piers 37 feet wide, with the bottle-neck viaduct, would have been utter waste of money for the chief port of Western Australia. It was fairly certain that had there been any sand-travel, Nature would in time have completed the work of connecting the harbour with the shore, beginning at the southern end. Mr. O'Connor's task had been a difficult one, because he had had to decide upon a harbour with an approach of more than 10 miles through a 6-fathom channel in sand in the open ocean. It would have been very valuable had information been given as to Mr. O'Connor's reasons for adopting the river in preference to the foreshore. The only light thrown on this point was the estimate of £800,000 approved in 1892, which in subsequent years expanded to nearly £1,500,000. At all events, the Swan River seemed now to have reached its development on the south-eastern side with about 5,000 feet of wharfage, while on the north side extension could be got only by reclamation on the foreshore, enclosing docks entered from the Swan River. It was unfortunate that Mr. O'Connor's and Mr. Palmer's proposals for the development of the south quays had not been carried out. As constructed the quays and sheds were cut off from the town by a railway sorting-yard. The dirt and smoke from the locomotives must alone depreciate greatly the value of the town properties adjacent. Nothing but standage should have been provided, say, for empties near the South Mole, and for fulls near the railway-

Mr. Bell. bridge, and the sorting-sidings should have been placed where the traffic from both sides of the river could have been assembled together—perhaps anywhere between Fremantle and Perth. Practically $1\frac{1}{2}$ million sterling had been spent in providing 12,000 feet of quays (equal to £121 per foot of quay), or, leaving out the outside jetties and the Rottnest light, the cost for 10,000 feet of harbour-quays might be taken as nearly £137 per foot. There was already an agitation to bring shipping up the Swan River to Perth—a step which would be of more value to the city of Perth than to the State; but the real problem was that the accommodation was limited to vessels not exceeding 30 feet draught, because with the exposure of the approach to Gage Roads, the navigation would presumably require in heavy weather at least 6 feet depth of water under the keel. Fremantle was therefore faced with the same questions as to depth which were pressing for settlement in Melbourne and Sydney. Not far south of Gage Roads was Cockburn Sound, selected by Admiral Sir Reginald Henderson, K.C.B., as the western naval base of Australia. This had a depth of 10 fathoms, and seemed to be the only natural site for a first-class port on this coast; but the sand-movements and the configuration of the sea-bed from Rottnest to Cockburn Sound must be clearly ascertained before either Gage Roads or Cockburn Sound could be fully developed as permanent national harbours. In view of the large expenditure and the limited accommodation at Fremantle, Mr. Bell considered that the best and most economical scheme for the establishment of a first-class port, unconditioned by existing interests and capable of the fullest expansion, would have been an artificial harbour on the foreshore, situated where the best access to deep water might be obtained either naturally or by dredging. As a case in point, he would instance the harbour of Burnie, in Tasmania, where an extension of the artificial harbour was about to be made from his designs. The present extension would provide 36 feet depth at low water, and the works were laid out with a view to expansion. The approaches were clear, with no outlying dangers, and the natural depth at the entrance of the complete scheme would be 42 feet at low water, with 11 feet range at spring-tides. The bottom was clay, with a covering of sand, and by reclamation of the foreshores with dredgings not less than 30,000 lineal feet of quays could be obtained, with depths up to 42 feet at low water, at a cost of £100 per lineal foot of quay, inclusive of breakwaters, sheds, and equipment. Taking the relative exposures into account, and the nature of the beds and foreshores, he considered that such a harbour, with a working-

depth of at least 30 feet, could be established at a similar cost at Mr. Bell. Gage Roads, and that it would be capable of being extended in depth and in area more readily and economically than works in the Swan River.

Mr. H. A. BLOMFIELD considered it to be a cause for satisfaction **Mr. Blomfield.** that Mr. Halligan had taken up his pen on the subject of bar harbours, how the bars were formed, and the remedy for the trouble. This branch of engineering was one of the most difficult to understand and one in which it was almost impossible to calculate results. In many cases more light needed to be thrown upon the subject. Tides, scour in rivers and estuaries, the set of the littoral and ocean currents, frequency or infrequency of floods, etc., waves, and wind-action, were all more or less important factors in the results of every interference with Nature which man attempted; and only by very close study of all these matters separately and collectively was it possible to arrive at a design for an artificial harbour which would ultimately carry out the aim of the engineer. Mr. Halligan attacked the scour theory, and in a country where there were no heavy floods during periods of 10 years or more, he had some right to do so, as it would be admitted that, unless a superabundance of upland water were present, the discharge would not clear the channel. The reason clearly was that in the ocean the sand had been stirred up and was in suspension, and the inrushing flood-tide carried it into the mouth of the river, where it was deposited. The ebb-tide, however, had not the same favourable conditions; as a rule it was, for months in a year and perhaps from year's end to year's end, a very gradual and easy flow, having neither waves nor tempest to aid it in stirring up the sand. No doubt the velocity increased to the ocean; but this velocity was only about equal to that of the flood-tide, and therefore he did not see that it could take out as much as the sand-laden water of the flood-tide brought in. Thirty years ago The Institution discussed a Paper¹ by Mr. Walter Raleigh Browne, M. Inst. C.E., in which the author argued that the flood-tide brought in more sand, silt, etc., than the ebb took out, and the velocity of the ebb-tide was stronger at the surface than in the centre or bottom; in fact that, while strong at the surface, it was slack at the bottom, and so even with an apparent rush out the particles of silt, etc., in suspension did fall and were deposited in the channel, thus not only not scouring it but in reality silting it up, because the scouring-power of any current depended

¹ "The Relative Value of Tidal and Upland Waters in maintaining Rivers, Estuaries and Harbours," Minutes of Proceedings Inst. C.E., vol. lxvi, p. 1.

Mr. Blomfield. wholly on its velocity at the bottom. The late Mr. Vernon-Harcourt, in the discussion on this Paper, admitted there was a good deal of truth in what Mr. Browne had said, namely, that the tidal water alone of the ebb was not able to remove the whole of the silt brought up by the flood. If, therefore, there were no floods or heavy rushes of upland water, might it not be concluded that during this period the ebb-tide did not keep the channel free by scour? And if this went on for 12 months, or, worse still, for years, then the theory of the efficacy of the ebb-tide scour must be abandoned. Mr. W. Smith also seemed to agree with this when he said, 21 years ago,¹ that the supposed effects of ebb-tide currents, guided by artificial works, should be more clearly proved from observation before the profession went to sleep over the theory; and that millions of money had been thrown away upon this time-honoured fallacy. The coastal current on the shore of New South Wales always went in the one direction, and the point was whether this current carried a quantity of sand in its course or not. With the 2-knot current always in the one direction, to the south, a very considerable quantity of sand must surely be always carried south by the current after heavy seas; and this water, given a turn obliquely on to the coast by the north-east winds, deposited its burden of sand on the beaches, and in the bays and river-mouths. This could be observed daily, as well as the denudation of sand from the beaches by the southerly or south-easterly gales; thousands of tons had been washed out by the heavy gales of October, 1910, and January, 1911. The beaches on the coast were always fed, in a gradual manner, by the north-east winds, the apparently unobtrusive action of which seemed to cause its subtle destructive power to be forgotten. One statement in Mr. Halligan's Paper, comparing (p. 142) the catchment-area of the Clarence River of 8,505 square miles, the largest in the list, with Crookhaven, of 40 square miles, the smallest, was misleading. Of the £462,000 spent on the Clarence River, £182,830 was the cost of the first scheme, most of which was discarded by Sir John Coode. Up to date, the expenditure was £283,419, but this had been devoted altogether to inside training-walls, and no breakwater work of the scheme had yet been carried out; nothing whatever had been done to help the bar, which was practically in its virgin state. With regard to Crookhaven, which had cost £9,000, this was one of the two entrances to the Shoalhaven River—occasionally the only one—and the Shoalhaven River had several thousand square miles of catchment-area. The

¹ Minutes of Proceedings Inst. C.E., vol. c, p. 201.

Crookhaven was not a river at all. The work here was almost entirely inside walling, and the breakwater had been advanced only into 8 feet of water at low water, just abutting on to a reef dry at low water; its beneficial result so far had been only to prevent the sand from the current and beach-wave action from entering the river. The comparison therefore did not hold good. From Mr. Blomfield's 5 years' experience as resident engineer on the Clarence River, he knew that the inside works were a decided advantage to the navigation of the river, and should not be disparaged; if the northern breakwater had been built to keep the sand out, there would, in his opinion, have been fully 18 feet of water on the bar at low water of spring-tides.

Mr. T. E. BURROWS felt that Mr. Halligan had taken a great deal of trouble in compiling the information contained in his Paper, and if not agreeing with the deductions made, he considered that the subject had been dealt with in a manner worthy of the consideration of harbour-engineers. The trend of the coast-line as a whole was north-east and south-west, and the offshore current was usually no nearer the land than about 3 miles. Steamers going north always kept inshore, while those going south were usually 3 miles out or more. Mr. Halligan's contention that there was a littoral-drift current from north to south along this coast-line was, in Mr. Burrows's opinion, erroneous. No current occurred on the littoral sufficient at all times to cause any serious or regular movement of material along the shore in one recognized direction. It was shown that along the whole coast the movements observed had been in contrary directions, varying according to the locality. Mr. Halligan also stated, as a reason for the alterations from what he considered the general set of the littoral current, that the headlands at the northern ends of beaches caused an eddying southerly current at these places. This reason was entirely insufficient, in Mr. Burrows's opinion, to account for the movement of sand and other material to the north at these places. As in other parts of the world, the controlling power over the littoral drift was the prevailing heavy winds (which in this case came from the south), causing heavy seas to stir up and move the material in a northerly direction. Where an entrance or beach was protected by a southern headland, the heavy seas and winds caused an eddying current to the north of such headland, and moved the material in the immediate vicinity to the south. Mr. Halligan stated that at the Richmond River, and at other places spread over the whole length of the coast, he had found that the travel of the sand, as Mr. Burrows suggested, was to the north. It was also to be noted that where an entrance was protected by a southern headland the depth of water at the entrance was much more stable than at places not so favourably

Mr. Burrows. situated. The Clarence River, Newcastle, Crookhaven, Bateman's Bay, Moruya and Wagonga were instances, and throughout the past 25 years, during which Mr. Burrows had been closely connected with the construction of harbour-works along the coast, he had found the bars there to show almost a continuously even depth of water. During the summer months the north-east wind was prevalent, but it was for the most part of a mild character that caused no heavy sea, and would have but very small effect on the current along the shore. It would be observed from the trend of the coast that all winds between and including the north and west points of the compass might be discarded, as they had no effect on the current or sea. The easterly wind had a very marked effect on the heaping-up of sand at the river-entrances. It blew directly on shore, and when a gale occurred from this direction (fortunately a rare occurrence in recent years) the ill effects at various ports were considerable. A south-easterly wind was the next most troublesome to harbour-engineers, and it was by gales from this direction, owing to their greater frequency, that most of the damage was done. Southerly and south-westerly winds also created heavy seas. Taking the whole action of the prevalent heavy winds affecting the sea and currents, there would be no possible doubt that the quarter from which most effect was experienced lay between south-west and south-east. The mileage of the winds from this quarter taken from the Author's Table was 30,567 miles, as against 14,898 miles of north-easterly wind. Again, taking the winds of more than 30 miles per hour, the proportion, discarding the offshore winds, was as 37.2 to 1.2. It was quite evident, therefore, that if the prevailing heavy winds controlled the movement of the littoral drift, then the movement must be from the south-west to the north-east. Mr. Halligan stated that these storms were of rare and uncertain occurrence, but their frequency was sufficiently obvious to Mr. Burrows to account for a considerable sand-movement to the north for the whole length of the coast. Where a river-entrance was protected from the action of the southerly winds, the movement of the sand at the entrance was very slight. Such entrances, from north to south, were the Clarence, Hastings, Camden Haven, Port of Forster, Newcastle, Shoalhaven, Bateman's Bay, Moruya, Wagonga and Bermagui; and though the induced or eddy current from the littoral drift set into the entrance from the north, it was an eddying current with very little power to cause any movement of the shore-material. The Clarence entrance was one of the most stable along the coast; the width on the bar was as Nature made it, but the inside channel had been diverted

and drawn in to 1,450 feet. Before this was done, in accordance Mr. Burrows. with the late Sir John Coode's design, the channel over the bar had a travel of about 2 miles to the north, and was uncertain in direction; and an inside crossing carrying only 7 feet at low water formed a terrible obstacle to the safe navigation of the port. By the construction of these inner works, the position of the channel over the bar had been fixed within narrow limits, and the position of the old crossing now carried more than 20 feet of water. So far from the sand being drawn into the entrance by the narrowing of the portals, there were now several million tons of sand less in it than when the existing works were completed. The breakwaters which Sir John Coode directed should be constructed first had not yet been commenced, and Mr. Burrows was confident that if these had been carried out in the first instance the entrance to this river would have required no attention from dredgers, except perhaps after one of the dreaded easterly gales. The Hastings entrance was also favourably situated as to its protection from the prevailing heavy winds, and was also a particularly stable one, usually carrying about 7 feet of water at low water. One short length of breakwater as an alignment to the channel on the southern side had been constructed, and the inner and adjacent portion of the tidal compartment was very wide and shallow. A crossing formed well inside the entrance, owing to the current of the flood- and ebb-tides being allowed to wander without direction. The width of entrance was practically as Nature made it, and there was little effect from any sand drawn into the harbour. Camden Haven and Port Forster were two minor ports where no serious attempt had been made to construct works which would affect the entrance. Some training-walls inside had been constructed, but these could not be said to have advanced sufficiently to control the current at the entrances. It was necessary, however, to draw attention to the fact that the eddying current from the main northward littoral current set south at both these entrances. The port of Newcastle at the mouth of the Hunter River was second in importance only to Sydney, and a considerable expenditure had been incurred there in the construction of training-walls and breakwaters. Protected by a southern headland standing well out, as well as by a south breakwater at the southern end of a large sandy bight (Stockton), the northward littoral main current, induced by heavy winds, was diverted under the lee of the headland and came back to the entrance along the northern beach as an eddy-current, being at this place strong enough to move material from the beach into the entrance. The other ports so protected were on the south coast, and although the movement of

Mr. Burrows. the sand in the vicinity of each was said to be from the north to the south, these entrances, as far as their bars were concerned, had a constant and almost uniform depth of water, and any trouble to navigation arose from the cross action of the flood- and ebb-tides shoaling the inner crossing. Mr. Halligan contended that in these places, through the entrance being narrow, the sand was induced by the action of the flood-tide to enter the port, and that the scouring action of the ebb-tide was not sufficient to remove it again. Mr. Burrows certainly did not agree with this statement, as all the entrances would have closed up long ago had this been the case; the ebb-tide scour had a function—and a very active one—in keeping the entrances clear of any sand swept in by the wave-action.

Another statement to which strong exception might be taken was that the upland water of the coastal rivers might be regarded as a negligible factor in the design of harbour-entrances. The late Sir John Coode, when designing the harbour-works for the entrances at the Richmond and Clarence rivers, was informed of the occurrence of high floods on these rivers—and rightly so, as up to that time these floods, although coming at uncertain intervals, had been large factors in the making and marring of the existing channels at the river-mouths. The designs were therefore so made as to carry a flood-discharge without creating a gorge, and although there had been an absence of high floods during the past 18 years, there was, in Mr. Burrows's opinion, no doubt that such floods would recur and the accumulated deposit in such entrances as the Richmond River would then be swept out to sea and removed by the offshore current, or deposited in deep water where it would give no further trouble. Mr. Halligan also missed the main action of storm-waves in stirring up the sand and other material at a greater depth than was done in fine weather. This material so loosened was carried farther along the coast obliquely and cast up on the shore. The Paper disparaged the value of the transformation effected at the Richmond River entrance by the construction of the harbour-works there, and as these works were the only ones which had been practically completed, Mr. Burrows proposed to show that the effect of the works had amply borne out the promise of the designer, who anticipated that 12 feet of water would be obtained here at low water. As pointed out before, little or no assistance had been gained from the expected occasional flood-scour for the past 18 years, and the works had been carried out during a period extending over about 22 years; this had allowed the lodgment at the entrance of an accumulation of sand that could be removed only by a flood or by expensive dredging. The minimum depth of 8 feet mentioned by the Author had

obtained for a short time after a heavy easterly sea had thrown up Mr. Burrows. the sand on the bar; but practically no dredging on the bar had been necessary, the ebb-tide scour having removed the obstruction in a short time. The position of the entrance had been fixed definitely; it previously travelled over a distance of 7,200 feet and was constantly changing its position, thus causing the inner channels to become tortuous and shallow. From being one of the most dangerous entrances and one frequently impossible to navigation—on some occasions for months at a time—it had become one of the safest on the coast, and ocean steamers had been able to enter and leave at night-time, as well as to negotiate it successfully at low water and half-ebb.

Mr. Halligan was unfortunate in his selection of the Manning River as a port to which he would apply his theory of a wide entrance. The designer of the harbour-works had made use of the northern rocky foreshore as a base for the root of the north breakwater, and the current was so trained that it would be carried along this concave wall and be always in the same direction. The ebb-tide scour was now only sufficient to maintain a channel 200 feet wide between the northern wall and the advancing southern spit. If the wide entrance were constructed clear of sand (a matter of impossibility in Mr. Burrows's estimation), or powerful dredgers were set to work to clear it, the up-river deposits would move down and fill the space so left, and necessitate constant dredging to keep it clear. The old order would return; and instead of having to clear a small area, as would be the case with Sir John Coode's design, the area requiring to be dredged would be doubled; indeed, the southern wall, as proposed by Mr. Halligan, would soon be entirely buried. A considerable quantity of sand was driven into this entrance by southerly gales; not permanently, however, as the ebb-tide was sufficient even here to cause the larger portion of it to be removed again, and the cutting-off of this supply of sand by the construction of the southern breakwater in the position designed by Sir John Coode would eventually make this entrance as safe for navigation as those carried out at the Richmond River had done. In dealing with the ebb-tide scour, Mr. Halligan had not mentioned that the time of ebb was 1 hour longer than that of flood, and that this alone would prevent accumulation of the sand in the entrance to any large extent. This effect was greatly added to by the discharge of the upland water; Mr. Burrows had seen in a moderate freshet in 1901 on the Richmond the run-out lasting throughout the whole of the 24 hours. The tide certainly rose and fell, but the surface current was strong enough to prevent any run in the direction of the flood-tide. He would

Mr. Burrows. also point out that when the tug-boat "Protector" was wrecked at the Richmond entrance in 1902, there was no local wind to account for the very heavy sea prevailing, the storm generating the sea being some distance off the land; but the south-easterly sea creating the ocean-current along the littoral was sufficient to carry wreckage and portions of the bodies of the drowned men about 50 miles north of the port. Mr. Burrows had been resident engineer on the south coast for about 15 years in charge of the harbour-works at Moruya and Bateman's Bay and other places, and subsequently district engineer for 2 years at the Richmond River, and during the latter portion of this time he had also had control of the Clarence River harbour-works; a portion of the coast, therefore, had been for a long time under his constant observation, and the harbour-works at the entrances named had been closely studied with regard to the movement of sand, as affected by ocean-currents and storm-waves.

Mr. Coode. Mr. A. T. COODE considered that it was not clear how the examples of Sydney, Botany Bay, and Jervis Bay, which Mr. Halligan cited to support his main argument in favour of wider entrances, could be made to apply to the cases of the Manning and Richmond rivers. The entrances to the former harbours remained stable apparently without any artificial works, while those of the Manning and Richmond rivers wandered up and down the coast so long as they remained in their natural state. It was difficult to believe that the determination of the width to be given to an entrance was such a simple rule-of-three sum as Mr. Halligan suggested, even though the application were confined to ports on the same coast-line. Thus, on the South African coast, the entrance to Durban Harbour would be found to have a width corresponding approximately with that given by Mr. Halligan's formula; but the success which had attended these works depended largely, it was understood, upon the dredging-operations, which according to his argument should not have been necessary. Again, at East London the distance between the mole-heads was greater than was required by Mr. Halligan's rule, so that there also, where successful results had followed dredging in conjunction with training-walls, dredging should not have been needed. At these two South African ports the tidal rise was very similar to that on the coast of New South Wales; the upland water had the same comparative insignificance, except on the occasion of freshets; and there was a considerable travel of sand. It would be interesting if Mr. Halligan would outline the methods which he would have adopted, apart from a wider entrance, to improve the mouth of the Richmond River. The criticism of Sir John Coode's

remarks on the ultimate movement of shingle, and the qualification Mr. Coode. which Mr. Halligan considered should be added thereto, appeared to be based only upon observations of the power of the littoral currents on the coast of New South Wales to transport sand.

Mr. E. A. CULLEN, from experience of the harbours and rivers of Mr. Cullen. Queensland, agreed to a large extent with the conclusions arrived at by Mr. Halligan as to sand-travel being the dominating factor. On the coast of the southern part of Queensland conditions were somewhat similar to those of northern New South Wales, save that the summer winds from the north-east quadrant were less pronounced, being, in fact, sea-breezes rising in the forenoon and usually dropping after sundown. The winds with greatest effect on sand-travel, therefore, were those from the south-east quadrant; and on flat, sandy beaches, with seas striking obliquely thereto, wave-movement transported large volumes of sand. The ocean-current referred to by Mr. Halligan was also experienced on the Queensland coast as far as the southern termination of the Great Barrier Reef, that was, at Breaksea Spit and in the vicinity, where shipmasters had reported a speed of 4 knots per hour at certain irregular periods; whether the current was experienced farther north outside the Barrier appeared to be uncertain. This current continued thence southward with speeds varying up to 2 knots per hour, according to season; yet, owing to the counter-currents produced by the contour of the coast and to the predominant winds, the general sand-movement was to the north. Mr. Halligan's remarks on the sand in the estuaries, on the New South Wales winds, and on the values and quantities of upland water, applied almost equally well to Queensland. The flood-tide was a potent factor in introducing sand or silt. In the Brisbane River, which discharged into Moreton Bay, with onshore winds the river-water was discoloured for 20 or 30 miles up, whilst during weather that produced no chopping wave-action on the bar the water in the river was clear enough for objects to be visible at a depth of 3 or 4 feet. Water impounded in the dry dock 18 miles up, and at rest for 24 hours, yielded a deposit which, when dry, varied up to $\frac{1}{8}$ inch in thickness: most of this was brought in by the flood-tide from the bar. Mr. Halligan's Table of ratios between width of entrance and tidal area might be added to by establishing the ratios between the cross-sectional areas of the entrances and the volumes of tidal water entering, and it would be interesting if these ratios could be furnished for the harbours cited. The river-entrances shown in Figs. 6, 7, and 8, Plate 5, would suggest that the entrance-works as executed had been designed without special reference to the

Mr. Cullen. factors that would be dictated by systematic regularization of each river as a whole. Perhaps Mr. Halligan could state the reasons for ignoring those factors, assuming that to have been the case. Taking into account the expenditure mentioned and the results obtained, it might be considered better practice to treat similar entrances by dredging alone; it would certainly be much cheaper in capital cost.

Mr. Deane. Mr. H. DEANE wished to express his concurrence in the main arguments of Mr. Halligan's interesting Paper, and in the conclusions drawn by him. He himself had paid considerable attention to the meteorology of eastern Australia and the discharge of the rivers, particularly those of the coast of New South Wales. The rainfall on the east coast of Australia, which was rather remarkable for its high average, was made up largely of heavy falls, occasionally amounting to several inches in 24 hours, while for long periods the weather might be exceedingly dry, during which time the river-flows very much diminished. It followed from this that the discharge from the rivers could not be depended upon, as a rule, to assist the ebb-tide in keeping harbour-entrances clear. It was therefore important to avoid any set of conditions increasing the strength of flow of the flood-tide which would bring the sand in. The entrances should be kept wider than ordinary practice would dictate, and in this way there would be no tendency for the travelling sand to enter, but it would be carried past outside. Mr. Halligan was of opinion that the movement of sand along the coast was effected by the agency of the littoral current, and not by storms and heavy seas. It was evident that currents of the velocity mentioned were not strong enough in themselves to translate the sand, and apparently the sand, having first been stirred up by a wave and carried up the beach, settled on its return in a position farther on in the direction in which the current was moving; the next wave repeated the operation, and so on.

Mr. Halligan. Mr. G. H. HALLIGAN thought Mr. King's Paper was particularly instructive and valuable as showing, by continuous charts, the sand-movements taking place from time to time, as the result of work done in a sandy entrance. This diagrammatic method was much more effective than masses of figures, though the contour-lines were rather close, and the result was rather confusing owing to the small scale to which the plans had had to be reduced; if one contour, say the $3\frac{1}{2}$ -fathom line, were selected and emphasized on each diagram, ideas could be fixed much more readily. Before discussing the plans it would be as well to state that at the Newcastle entrance

there was an ocean-current with a normal velocity of $1\frac{1}{2}$ to $1\frac{3}{4}$ knot Mr. Halligan. per hour. It flowed from north to south, except during southerly or south-easterly gales, when the surface direction was retarded and sometimes even reversed during the period of the gale. These occurrences, however, were so infrequent and of such short duration that their influence upon the ocean-current might be neglected. The result of the current flowing always in one direction was to convey sand along the beach from north to south until it arrived at the entrance to the estuary; the flood-tide then conveyed most of it in, and the ebb took some of it out again, depositing the greater part of it in the form of a bar at the river-mouth; from there the current was continually taking part of it into suspension and carrying it away to the south. When the island of Nobby's was joined to the mainland, and the south breakwater was built, it formed a trap to catch this sand, which had extended farther and farther out, on the north side, as the south breakwater was continued. The object of the north breakwater was to stop this sand from entering the estuary, but a cursory glance at the $3\frac{1}{2}$ -fathom contour-line would show that this result had not been achieved. There was practically no change in the $3\frac{1}{2}$ -fathom line between 1896 and 1905, notwithstanding the fact that 2,500 feet of north breakwater was built during that time; the reason being, in Mr. Halligan's opinion, that the ocean-current still conveyed the same quantity of sand to the entrance, and that it was still met by the same trap—the south breakwater. After 1903 dredging was carried out, and the following figures showed the quantity of sand dredged from the Newcastle bar between the 1st July, 1904, and the 30th June, 1910:—1904–5, 60,100 tons; 1905–6, 156,340 tons; 1906–7, 183,075 tons; 1907–8, 399,050 tons; 1908–9, 573,050 tons; 1909–10, 506,404 tons; total, 1,878,019 tons. The result of this dredging was shown in Figs. 9 and 10, Plate 6, and unless dredging was continually kept up, the entrance must again become as bad as before. It was now proposed to extend the north breakwater another 420 feet, in the hope, as Mr. King said, that the sand-drift would be kept within easy control; but no reason was given for this optimism. The same quantity of sand must be brought to the mouth by the southward ocean-current, and as it did not heap up behind the north breakwater it must cross the entrance; it could only do this by entering with the flood-tide and in part coming out again on the ebb, and the same old trouble would have to be overcome by resorting to the expensive process of dredging. Unless the north breakwater was continued so far out as to be farther east than the south wall, and the entrance was

Mr. Halligan. made so wide that the velocity of the flood-tide would be less than that of the ocean-current, Mr. Halligan could not see how any improvement in the existing state of affairs was possible. It would probably be cheaper to stop building the breakwater, and accept the annual charge for dredging as inevitable, than to extend the north breakwater as proposed.

Major Harts. Major W. W. HARTS, of the Corps of Engineers, U.S. Army, remarked that Mr. Halligan's long experience of 35 years on the east coast of Australia would naturally enable him to speak with much authority as to the physical conditions of that locality; and his study of ocean-currents, sand-movement, winds, and tides during his connection with the Government of New South Wales had given him excellent opportunities for drawing conclusions as to the harbours and bars along the coast. Some of these harbours were singularly like several of those on the coasts of the United States, where the entrances were more or less obstructed by sand-bars, of which Major Harts had some knowledge. It was, therefore, with considerable hesitation that the following comments on Mr. Halligan's conclusions were submitted. His statement that sand was brought into the harbour on the flood-tide, and then deposited so that all of it could not be removed by the ebb-tide, led to the belief that he regarded all harbours on sandy coasts as sand-traps into which, little by little, sand was carried in suspension, never to be removed by other currents at ebb-tide. The fact that many harbours of considerable size on sandy coasts had remained open and in a condition of equilibrium for many generations, and perhaps centuries, would indicate that there was something faulty in Mr. Halligan's conclusions, and that there must exist in all such cases some balance between the forces tending to fill up the harbour and other forces tending to enlarge and deepen it. That this balance was an absolutely fixed one few engineers would dare to say; but that it existed only few would probably feel inclined to deny. When the depth over the bars of harbours in their natural state was insufficient for navigation, the controlling works placed by engineers in the entrance introduced a new element, which disturbed the old equilibrium and produced new conditions, giving rise to a new balance between the contending forces, the results of which, if the works had been properly designed, were a greater depth and better facilities for navigation. If greater depth were desired, experience at many harbours showed that a narrower outlet would be necessary, to assist in securing the conservation of the energy of the ebb-tide and a greater control of its field of action. Mr. Halligan's assumption that greater depths would

follow a widening of the entrance, and consequent reduction of the Major Harts, velocities of the currents, seemed contrary to experience elsewhere. Major Harts agreed that the upland water was of no importance in the average case, but he felt inclined to differ from the statement that the effect of the velocity of the ebb was not somewhat greater than that of the flood. The greater power of the ebb-tides for transporting sand, and thus improving the conditions within a harbour and at its entrance, had been relied on by many eminent engineers; and their numerous works showed that their views had not been sufficiently erroneous to cause disappointment. As a harbour filled from a limitless supply of water on the rising tide, the water entered from the sea without being confined to any special direction, and after entering it spread out in all the irregularities of the bay and its many indentations, creeks, and channels. Not only was this water not directed as it entered the harbour, but it flowed with a constantly decreasing hydraulic radius until the harbour was filled, so that the currents, after they passed the entrance, diminished in velocity. When the tide fell, the water flowed out with a constantly increasing hydraulic radius at a constantly increasing velocity, and the direction of the flow was controlled by the banks and channel; so that it seemed reasonable that its action at the entrance should be more effective than on the flood-tide. The favourable results already achieved in the improvement of bar harbours were usually attributed to direction of the ebb flow towards the proper place, and to prevention of any waste of energy as it emerged from the outlet. The great depths that had been found inside the bar in many harbours indicated that this process of ebb scour was more nearly what actually happened, than that the harbour was being gradually filled from without by sand that could not escape. The crescentic shape of the bar obstructing many entrances indicated that the ebb-tide, laden with sediment, deposited its load only when its velocity had been checked by the quieter water in the ocean. If these views were correct, it would seem that Mr. Halligan's statement that breakwaters or jetties should be as far apart as possible could not be accepted without question, but that there was an artificial width for the entrance at each harbour which would produce a new and more favourable equilibrium, with greater depths and with lessened tendency to shoal. In the case of Newcastle Harbour, the shoaling which had taken place between the breakwaters indicated that the balance between the filling and eroding forces had not yet been properly secured. Major Harts's comments were necessarily based

Major Harts, on descriptions and maps of a coast with which he was unfamiliar, but the successful application of certain fundamental principles to the improvement of American harbours led to the conclusion that some of Mr. Halligan's statements were not sufficiently supported for general acceptance.

The map of the Newcastle entrance in 1866 (Fig. 2, Plate 6), before any improvement had been attempted (with the exception of the breakwater connecting Nobby's Island with the shore), showed the harbour to be similar in many respects to the usual entrance through a sandy coast, where the apparent effort of the littoral sand-bearing currents and storms was to close the entrance with sand, against which the tidal flow through the bay struggled to scour sufficient channel-capacity to accommodate the inflow and outflow of the tide. Although all of the forces at work about this entrance were not known, Mr. Halligan's Paper demonstrated very convincingly that the littoral current across the mouth was from north to south. Although severe storms came sometimes from the south-west and south-east, the prevailing winds came from the north and north-east, and usually set up a current along the shore in their direction. This current aided to some extent the permanent ocean-currents flowing in the same direction, and it was therefore natural to expect that the main sand-movement would also be in that direction. A study of the various maps illustrating Mr. King's Paper tended to confirm this conclusion. Upon the construction of the south breakwater it appeared that this sand-flow was more or less interrupted, and, although the tidal currents kept the channels open, there was apparently a constant tendency to deposit sand on the north spit, due to such interruption, and thus to encroach upon the channel. That might serve to explain, at least in part, the deflection of the line of deepest water toward the south in the years 1887-1909, an action that was still continuing, judging from Mr. King's reference to the tongue of the 5-foot contour, shown by the 1909 map. From an examination of the map of 1896 this action seemed to have been useful in contracting the entrance and directing the tidal currents to such an extent that the least depths over the bar had been upwards of 24 feet at that time. Then, too, there were indications several years before, as shown by the map of 1887, of the commencement of a "swash" channel, which by 1903 had deepened to more than 30 feet opposite the end of the north breakwater, thus permitting the escape of a considerable volume of the ebb-currents and consequent shoaling in the channel. All indications pointed

to the conclusion that the north breakwater should have been built first, and the channel have been narrowed thereby, to increase the velocities opposite and beyond the east end of Nobby's Island. Until this breakwater was long enough to cut off entirely the sand-movement into the entrance, or to divert it sufficiently far out to enable the cross currents to carry it by the entrance, there did not seem to be much hope of relief from the expensive dredging now found necessary. The construction of the north breakwater along the north spit had apparently been beneficial in protecting the entrance to some extent, but the decrease of depths between the north and south breakwaters would indicate that the ebb-currents of the bay had not been concentrated enough to control this tendency. Further, the entrance opened now to the north-east, thus inviting a flow of sand into the entrance instead of across the ends of the breakwaters. It would therefore seem that increased depths by natural erosion could not be expected unless the north breakwater were extended and the entrance were further narrowed. It was to be regretted that the physical features of this entrance were not more fully described, as the foregoing conclusions might very likely be modified by a closer study of the natural characteristics of the entrance.

Mr. T. W. KEELE observed that if Mr. Halligan were correct in his facts and the deductions he had derived from them as the result of 35 years' experience in the service of the New South Wales Government as a hydrographic surveyor, then it would be quite evident that engineers, the world over, had been designing and carrying out harbour- and river-improvement works on entirely wrong principles; and the waste of time and money through their neglect and incompetence must be enormous. It would be interesting to see how far Mr. Halligan's facts were in accord with the observations of others. In his remarks on monsoonal ocean-currents, he laid great stress on the influence of the north-east wind, and quoted from Mr. Dannevig's Paper in order to prove that it was the prevailing wind. The Table he relied on showed the average travel of the winds that blew from the cardinal points each month, from which he concluded that "with the exception of the westerly winds which have no great influence on the coast of New South Wales, the north-easterly are the prevailing winds, as regards number, wind-mileage, and duration." This was all very well as far as it went, but something more was required in order to determine the influence of the winds, and their effect upon ocean-currents and sand-movement. Mr. Dannevig had calculated the final resultant power for each month, and for the year, in order to

Mr. Keele ascertain the total "drift energy" which he had set forth in diagrams. He said :¹

"It will be noted that from November to March, the winds blow persistently from the sea and during May to September in an almost opposite direction ; April and October are peculiar to themselves, and of a neutral character. . . . The yearly resultant or normal drift is interesting as regards direction and extent, it is in accord with the meteorologist's contention as regards the general ingress of the lower atmospheric layers from the pole to the equator, but it is contrary to a popular idea which is based upon the importance of prevailing winds (with us from the north-east). The yearly resultant coincides to within a couple of degrees with the local magnetic deviation, and its relationship to the coastline might be noted."

With regard to the yearly resultants Mr. Dannevig said :²

"In comparison with their average—the yearly normal—they show individually considerable differences, yet with one or two exceptions they all point north with an easterly or westerly tendency as the case may be."

This was sufficient to show that the resultant "drift energy" of the winds and of the sand-movement was to the north, and not to the south, as stated by Mr. Halligan, and that the westerly winds must have a considerable influence on the littoral current from May to September, during which it was pushed out seaward, its effect on the sand-movement thus being practically nothing. When the winds were reversed and blew from the sea, although the littoral current was pushed over against the shore, its effect became nullified by the strength of the dominant southerly winds, which, as Mr. Halligan himself had shown, retarded and ultimately reversed the littoral current, with acceleration of the northerly set existing close in shore. In view of this, it would have been interesting to know what the late Sir John Coode would have thought of Mr. Halligan's suggestion that his statement that "the ultimate movement of shingle is always to be found in the same direction as, and never against, the heaviest seas" should be qualified by the addition of the words "on a shore where there is no continuous ocean-current in one direction." Mr. Halligan also sought to show that the sand in the estuaries must have come from the sea-coast, and have been carried in by the flood-tides. There could be no doubt that this was so to some extent ; but not exactly in the manner he would infer. Before any works were commenced, the rivers, during periods of drought, were almost closed at their mouths by the sand-spits making over towards the headland on either the north or the south side. These spits extended for miles in some instances,

¹ Proceedings of the Royal Society of New South Wales, vol. xli, p. 37.

² *Ibid.*, p. 39.

where the rivers ran almost parallel with the coast in their lower courses; but during the continuous wet seasons, lasting several years, when the annual rainfall over the watershed was persistently above the average, the rivers were in a constant state of fresh or flood, and the sand-spits and beach-terraces were cut away for a mile or more from the mouth, breaches in the beach-terraces being made at other places. On the subsidence of the floods, the entrances and breaches in the beach-terraces, being wide open and fully exposed, were at the mercy of the sea during onshore gales, and enormous quantities of sand were driven in by the waves. Sometimes, as at the Tweed and Macleay rivers, this caused complete choking of the lower part of the river for several miles, and compelled it to make a new outlet. On a recurrence of drought, the sand-spits gradually made up again, and small freshets, when they occurred, had to cut their way in tortuous courses along the old channels to the original mouth, or, in cases where they could not do this, an entirely new channel was formed, leaving the old one permanently reclaimed, and ultimately raised by wind-blown sand to the level of the surrounding country. When the harbour-works were commenced, there were large accumulations of sand which had been driven in from the sea in the manner described, and, as the training-walls progressed, the sand became imprisoned: it had lain there ever since, awaiting the flood-season, which would assuredly recur, when it would be swept out bodily into the ocean. The channels having been permanently fixed and protected by walls, no further breaches could be made, and although during droughts a tendency for bars to form at the mouths would recur, and where works were unfinished some sand might be driven inside for a short distance, the conditions were altogether different from what they had been originally. In an improved river like the Richmond or the Hunter, no sand would ever again be forced far into the estuary. The extensions of the breakwaters well out into deep water had demonstrated that, even at the termination of one of the most protracted droughts ever experienced in the State, the rivers were able to clear themselves with little assistance from dredging. The amount of dredging required during droughts would depend entirely on the depth required to be maintained at the entrance.

Under the heading of "Tidal Currents" Mr. Halligan made some extraordinary statements, which showed conclusively that he had yet to learn a great deal about the action of the tides and the influence of the upland water upon them, and their combined effect upon sand-movement. He was evidently unaware of the fact that,

Mr. Keele. although he might have found the velocity of the ebb- and flood-currents at certain times to be equal, the ebb ran out longer than the flood; and that, when it ceased to do so, the entrance shoal ultimately closed up entirely, and it could be reopened only by a flood. This was the invariable action of any river in its natural state during periods of drought, or when the fresh water had been abstracted from the main channel, either by diversion into a channel having another outlet, or by impounding the water in large reservoirs in the uplands. The fact that a river kept open at all was a demonstration of the superior influence of the upland water—no matter how small the quantity might be—on the ebb-tide, thus enabling it to counteract the influence of the flood-tide. Mr. Halligan's observations on the relative densities of fresh and salt water appeared to have been of a superficial character, otherwise he would have noted a fact ascertained long ago, that the flood-tide, in its passage up a river, pushed its way along the bottom owing to its superior density, and thus raised the fresh water above it; the latter continued its flow to the ocean, the tendency being to make the gradient steeper towards the mouth while the ebb was running out, and to increase its scouring effect. It was evident, therefore, that Mr. Halligan's advice to engineers to eliminate the area of the watershed, and the supposed scour of the ebb-tide due to upland water, from the design of an entrance with a view to improvement of the harbour, would deprive them of the dominant factors, and the result could not be other than disastrous. It was so long since a succession of wet years had been experienced on the east coast of Australia, that Mr. Halligan appeared to have become pessimistic, and to view everything from the standpoint of drought. It was a fact that since 1893 there had been a serious decline in the rainfall of the coastal region of New South Wales; for 17 years no serious floods had occurred, and very few freshets, the rainfall having been almost continually below the average. During the 5 years ending in 1893, however, when the rainfall was continually above the average, the rivers were constantly swollen, and it was during that period that most of the works were commenced, and the training-walls and breakwaters enclosed large areas of sand in the manner already described.

With regard to Mr. Halligan's contention that wide entrances were required, and not the narrow ones which so many engineers had advocated in the past, Jervis Bay, to which he referred on p. 141, was a land-locked bay with no river or considerable stream running into it. It had an area of 39 square miles of water 30 feet or more in depth, the 5-fathom contour-line running close to the

shore all round the bay. Mr. Halligan seemed to be quite unaware Mr. Keele. of the true cause of the bay constantly maintaining its depth, namely, that during onshore gales it became surcharged with water, forced in by the heavy seas driven by the wind, and held up against the shore at a considerable height above the mean level of the ocean. The configuration of the coast-line on each side of the entrance presented a concave outline to the sea, which tended to produce a gorging effect on the waves, and materially assisted in surcharging the bay. In the effort to restore equilibrium, an under-current was generated from the shore all round the bay, which, running along the bottom radially towards the entrance, became concentrated there, with great scouring effect. That this action was taking place was shown by the contour-lines on a chart of the bay being wrinkled or corrugated, the furrows all pointing towards the entrance. The depth at the entrances of Botany Bay and Port Jackson was maintained in a similar manner, and a similar action was taking place at every bay and indentation along the coast, whether land-locked or otherwise, or whichever way it might be presented to the onshore winds, whether from the north, east, or south. This action alone accounted for the ability of these bays to clear themselves of sand. It was astonishing that Mr. Halligan could not see that an analogy was impossible between the natural harbours he had mentioned and tidal rivers subject to floods. Mr. Keele had already explained what the action was in such rivers under drought and flood conditions, and it was unnecessary to labour the subject further than to say that, in his opinion, the "banks of marine origin" which existed inside the entrances and were so much objected to under present conditions by Mr. Halligan, would, under the conditions advocated by him, certainly expand, both horizontally and vertically, into excellent sheltered beaches, where surf-bathing might safely be indulged in; but the shipping would unfortunately be eliminated from the harbour. In conclusion, Mr. Keele hoped that the authorities would permit of no departures from the designs of Sir John Coode, and other marine engineers of experience, on the lines advocated by Mr. Halligan; in his opinion such departures would only end in complete failure.

The Institution was indebted to Mr. Palmer for another interesting Paper on an important engineering work carried out by the enterprise of the youngest of the Australian States. His excellent Paper on the Coolgardie waterworks in 1905 had placed the members under an obligation to him, and it was fitting that he should have undertaken the duty of placing on the records of

Mr. Keele. The Institution the history and details of this second of the great works which constituted a lasting monument to the foresight and ability of his late chief, Mr. C. Y. O'Connor. It was now evident that a wise decision had been made to provide a harbour by solid works at the entrance to the river, rather than an outer harbour connected with the shore by an open viaduct. Had the latter scheme been adopted, it would have been found, even at the present time, too small to accommodate the shipping that traded to the port. Thus, the arrivals at the port of Fremantle in 1894 amounted to 406 steam or sailing ships with a total net registered displacement of 337,820 tons; by 1904 these figures had increased to 845 ships with a displacement of 1,517,791 tons; and in 1909 the figure was 1,573,757 tons. Mr. Palmer stated that the present harbour could, at moderate expense, be made to accommodate 50 per cent. more shipping than in 1904-5. The time was not far distant, however, when even this limit would have been reached, and consideration must then be given to a proposal submitted to the Government of Western Australia in 1904 by Mr. Keele, namely, to remove the railway- and road-bridges shown in Fig. 3, Plate 7, provide for an efficient steam-ferry service for the road-traffic between North and South Fremantle, make connection between Fremantle and Perth by a railway on the south side of the river, and then open up the Swan River by dredging as far as Freshwater Bay. Probably the general public of the State—and certainly most people outside of it—were not aware that the State possessed, inside the bridges, a waterway capable of being opened up by dredging into a splendid channel, gradually expanding at Freshwater Bay and Melville Water into a land-locked harbour, which even now embraced an area of $3\frac{3}{4}$ square miles with 18 feet or more at low water, $2\frac{1}{4}$ square miles with 24 feet or more, and $1\frac{1}{2}$ square mile 30 feet to 70 feet deep at low water. This would be sufficient, if some shoals were removed, to provide a safe and sheltered anchorage for the whole fleet of war-vessels on the station, in addition to providing wharfage for all shipping likely to visit the port for a great many years to come. The present proposal was to dredge a channel 350 feet in bottom width, and not less than 30 feet in depth at low water, up to the deep water at Freshwater Bay: this channel was to be subsequently widened when required to the full extent, which would be nowhere less than 700 feet in bottom width, expanding to 1,000 feet or more as the deep water was reached after passing Rocky Bay. The sides of the channel were to be lined with a rubble wall $8\frac{1}{2}$ feet high along low-water line. The area between the walls

and the shore-line was to be reclaimed by the dredgers, and as the Mr. King.
Government already possessed about five-sixths of the frontage to the river, the reclaimed areas would, in the future, be exceedingly valuable. This proposal was the result of an investigation made by Mr. Keele, at the request of the Government of Western Australia, to determine the best site for a graving-dock. Seven sites had been proposed at one time or another, three of which were within the present basin below the bridge, while the others were between the bridges and Rocky Bay. None of these sites, however, was considered by him to be so suitable as a new site which suggested itself at Butler's Hump, on the western shore of Freshwater Bay. The total cost of the complete scheme, to include the dock and naval station, was £1,460,000, and he claimed that it fulfilled all the conditions laid down by the Admiralty, namely:—a depth of water sufficient to allow of docks and basins being placed so low as to admit at low water of spring-tides the largest naval ships at what might be considered "damage draught"; spacious and well-sheltered anchorage, of an area sufficient to accommodate a large number of vessels; natural protection from an enemy's fire; a central position on the coast-line with ready and convenient means of communication with manufacturing districts particularly, and with all parts of the country generally; and finally, close proximity to a well-protected watershed or harbour. The proposal had remained in abeyance up to the present time, probably on account of the interference which would result with direct communication between north and south Fremantle by the removal of the bridges; but no parochial consideration should have been permitted to stand in the way of opening up the wide and deep water of the river to navigation, thereby conferring directly and indirectly on the whole community benefits which could not be overstated. Mr. Palmer, made no reference to the question of the graving-dock, which, as appeared from the annual reports of the Fremantle Harbour Trust, it was finally decided, in 1908, to construct at Rous Head, at the commencement of the South Mole, and opposite the 6,000-foot mark shown in Fig. 3, Plate 7. The necessity for a work of this description was no doubt felt severely, but the site selected would certainly be found inconvenient in the future, situated as it was right at the entrance to the basin. The docking and undocking of vessels would be a serious inconvenience to shipping entering or leaving the port, and, notwithstanding any breakwind which might be erected, the exposure during onshore gales would be exceedingly great: moreover, the dock would be fully exposed to the direct-aim fire of an enemy's guns.

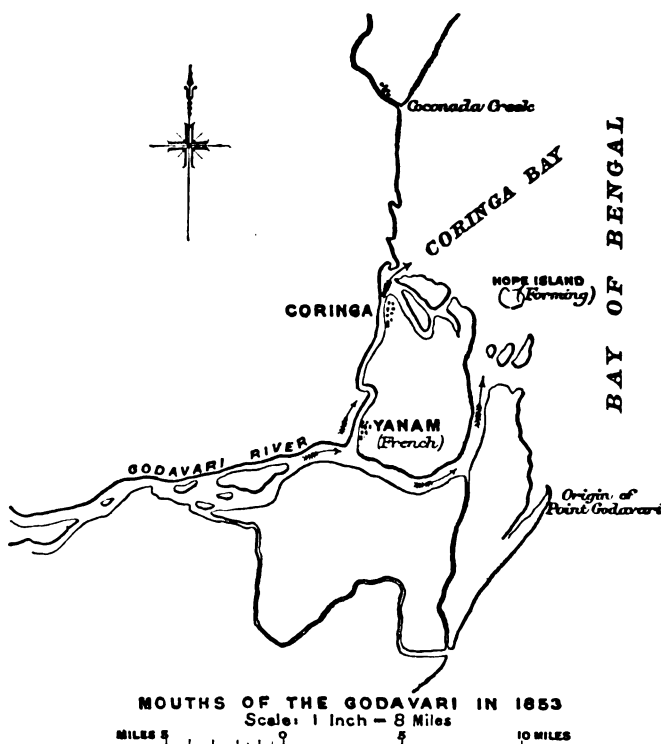
Mr. King. Mr. C. W. KING considered that Mr. Halligan's Paper contained very valuable data on questions connected with bar harbours on the New South Wales coast, and would be of great assistance in considering the design of future works. The inflow of sand into rivers and harbours should be considered in conjunction with the reduction of wave-range. To solve the former question completely would, in some instances, defeat the solution of the latter. As Mr. Halligan remarked, each case must be considered on its merits. With regard to the proposed ratio of 1 foot of entrance-width to 4 acres of tidal compartment, he would suggest that the discharge-area of the entrance-channel, and also the effect of the waves in stirring up the sea-bed, would be of grave importance in determining the position of the outer works. The mean tidal current through the Newcastle entrance was about $1\frac{1}{2}$ knot per hour during spring-tides. Adjacent to the north breakwater this current was accelerated to 2.4 knots per hour, as shown by recent current-observations. The fact that these velocities were greater than that of the ocean-current clearly accounted for the inflow of sand now being contended with. Operations, however, were in contemplation, and partly in progress, to increase the maximum depth from 23 to 30 feet or more. These operations, when completed, would increase the discharge-area, and incidentally they should decrease the mean velocity of the tidal current and the inflow of sand. He had observed that at Newcastle the inflow of sand was very much greater during heavy weather than in smooth, owing, as Mr. Halligan said, to the storm-wave tearing down the beaches and conveying the sand seaward. There was also the action of the wave on the sea-bottom in the shoal water adjoining the coast. Thus, if the tidal current was in excess of the ocean-current, the inflow of sand naturally increased during stormy weather. He was of opinion that if the entrance possessed sufficient discharge-area to reduce the tidal velocity below that of the ocean-current, the width and depth being proportioned as the consideration of each case might determine, then the breakwater works constructed on such principles would largely reduce the inflow of sand caused by the action of currents. A neutral line would also be reached, seaward of which the wave-action would be at such a depth that the disturbing force on the sea-bed would be reduced to a minimum, and landward of which the wave-action would disturb the sea-bed and cause the heaping up of a bar. Observations made by Mr. KING on a sandbank 29 feet below low water, and exposed to waves 27 feet high, showed no alteration in depth after a heavy gale, and he considered that the neutral line would be in the vicinity of 5 to 6 fathoms. The question of diminishing the wave-range was also of

great importance in harbours with wharfage and moorings immediately adjacent to the entrance-works, and in such instances he considered that a proportion of sand-dredging must frequently be provided for, to allow of the diminution of range. Thus, at Newcastle a wider entrance than that shown would allow the swell to enter still more freely, the distance between the harbour and the extreme entrance-works being too short to allow for sufficient wave-traps to decrease the range to a safe limit. In many river-entrances the wharfage lay miles away, and under these conditions the range inside need not be taken into consideration. In such cases Mr. Halligan's proposal merited very careful consideration in the design of future works, with a view to overcome the tendency of the sand to travel up-stream and to form the crossings which appeared so frequently on the rivers mentioned.

Mr. J. C. LARMINIE thought that a few particulars of a part of the Coromandel coast where the conditions were very similar to those prevailing at Newcastle, although on a smaller scale, might be of interest. The port in question was Coconada (about 300 miles north of Madras), second only in importance to Madras on the east coast of India, and with the best and safest anchorage in the Bay of Bengal. Following the successful completion of the Godavari irrigation-works and the extension to Coconada of the East Coast Railway (about 1894), the trade of the port had increased enormously, but all merchandise had to be conveyed to and from the shipping in open boats of shallow draught. The anchorage, about 5 miles out in a north-easterly direction, was well sheltered from east to south by the peculiar long tongue of land known as Point Godavari (*Fig. 4*). Its formation was comparatively recent, and was entirely due to the silt-deposits from the Godavari River, combined with the travel of sand northward due to the southerly littoral currents. An old survey of about the year 1853 showed that, at that time, the Point had only just begun to form (*Fig. 3*). The trend of the mouths of the Godavari was northward, as shown, with the result that the vast quantities of silt brought down in floods had led to the shoaling up of Coringa Bay and the formation of numerous islands (*Fig. 4*); while the discharge of the silt-laden water seaward, meeting the littoral current and the sand travelling northward, had led at the same time to the gradual formation and growth of this long low spit, also in a northerly direction. A map of the bay about the year 1891, taken from the Marine Survey, was shown in *Fig. 4*. By that time Point Godavari had reached its present northerly limit; but meanwhile its effects on the bay within had been marked and peculiar, being precisely similar to what might often be noticed on

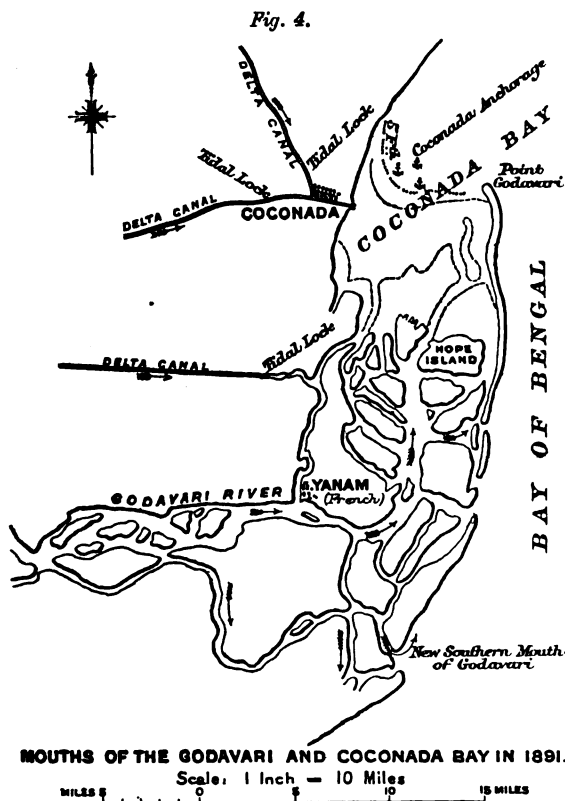
Mr. Larminie. the down-stream side of a river-groyne placed at an unsuitable angle; that was to say, scour was set up in the rear, owing to the reversed direction of the river-current. In this instance the tidal and littoral currents impinging on the eastern face of Point Godavari swirled round the point into the bay southward and westward, where, meeting the sandy coast, with its south-westerly trend, they originated the littoral current in that direction and its accompany-

Fig. 3.



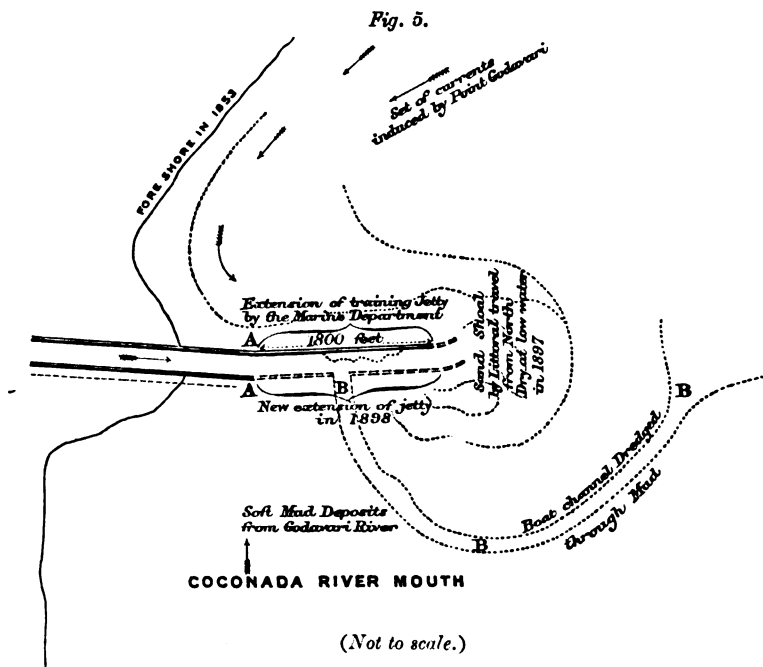
ing travel of sand, which was the origin of all the trouble at the mouth of Coconada River. In regard to the bay generally, this action resulted in an improvement and general lowering of the bed towards the coast and southward. The so-called Coconada River was originally nothing but a salt creek, extending inland in two branches, which carried off some of the Eastern Delta drainage. In its upper reaches, however, about 4 miles and 2 miles from the coast respectively, were situated the terminal locks of two of the Delta

main canals, whereby communication was established with the richest Mr. Larminie. producing portion of the district. It consequently became necessary from an early period to maintain communication between the river and the anchorage. Wharves were constructed, the river-slopes were revetted and the river-mouth was fixed by short lengths of training-jetties. The usual result—an advance of the foreshore—followed, accompanied by the formation of shoals and a bar across the river-



mouth, involving constant dredging, with the imperfect appliances then available. Up to about the year 1876 the work was in charge of the Public Works Department, but it was then transferred to the Marine Department. At this time the right and left training-jetties were of about equal length, terminating at A A (*Fig. 5*). With the idea that, by extending the north jetty, the travel of sand along the beach from the north would be checked and

Mr. Larminie. excluded from the river, the Marine Department gradually carried out this work, the result being that, by 1896, the trade of the port had become seriously hampered by the formation of the shoal indicated (*Fig. 5*). This blocked the river-entrance completely, and, overlapping the channel right round to the south, necessitated the dredging of the circuitous channel B B. At this stage, Mr. Larminie was deputed by the Government of Madras to report



on and devise a remedy for the inconveniences complained of by the mercantile community. The gist of that report was that the state of affairs was entirely due to the extension of the left jetty about 1,800 feet in advance of the right jetty; that so long as the ends of the two jetties had remained nearly in line, the scour of the ebb-tide had maintained a fairly deep channel, which was even still maintained, down to the present termination of the right jetty; and that, were the latter extended to the same length as the left jetty, the scouring effect would then be concentrated on the bar. The proposal was approved by Government, and the results had fully justified it; for, no sooner had the right jetty been extended and the gap at B closed, than a marked improvement in the depth on

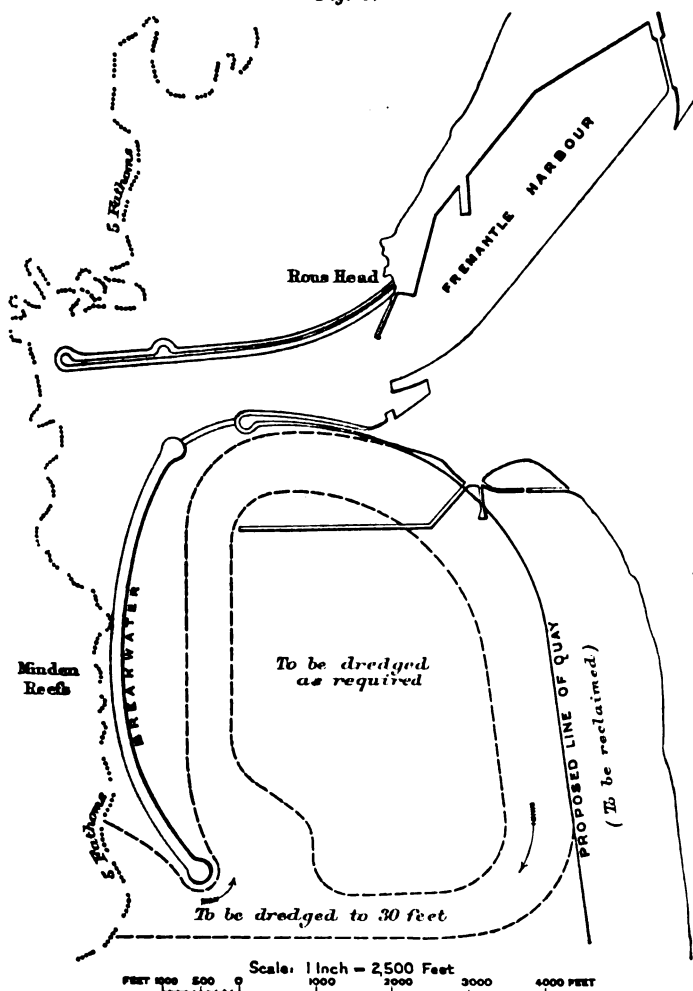
the bar took place. At the same time he urged that, no matter what proposal was adopted, the ebb-tide scour, unaided, was insufficient to keep the bar clear, and that the provision of a suitable suction dredger was imperative. At Coconada the tidal range was about $5\frac{1}{2}$ feet, while the basin within the bar was simply that comprised in the 5 miles of canalized creek before alluded to. The formation of Point Godavari would seem to confirm Mr. Halligan's contention (p. 136) that sand-movement was chiefly due to the littoral current, aided, as at Point Godavari, by the south-west monsoon prevailing from May to September, which, owing to the trend of the east coast along the Northern Circars, became a long-shore wind. As already noted in regard to the extension of the Coconada right jetty, Mr. Larminie's experience of ebb-tide scour was that it was by no means a will-o'-the-wisp as contended by Mr. Halligan; but the latter's deductions in regard to the connection of flood- and ebb-tide currents with the littoral current appeared to be very sound, though much would depend on local conditions. A narrowed entrance, no doubt, would head up the flood-tide, causing increased inward velocity; but this increased inward velocity would be some little distance within the entrance, removed from the coastal current, on which, in consequence, it could exercise but little draw. On the ebb, the reverse would take place, the discharge being delayed and scour over the bar ensuing, during a portion of the ebb-tide at all events. This was the explanation of what had actually occurred at Coconada, where the sand accretion shown along the left jetty had been actually removed by the ebb-tide scour. In the three instances of wide entrances given by Mr. Halligan, as he remarked, the north heads projected eastward considerably; the littoral current and sand-drift, therefore, were carried well clear of the entrances. It would require a very considerable indraught to divert the sand-travel from the southward, and consequently the entrances might be narrowed, if necessary, without injurious effect. The sand deposit on the north-east shore of Botany Bay might be from the Cook River. From Mr. Halligan's description it was not easy to infer whether it was really from this source or was carried in during south-easterly gales. As already remarked, there was also some similarity between the conditions at Coconada and those at Newcastle, as described in Mr. King's Paper; at all events, the latter port furnished conclusive evidence of the value of training and a narrowed entrance. Taking 1 foot of entrance-width for every 4 acres of tidal compartment, the width should be 2,570 feet, against an actual width between the breakwaters of 1,200 feet; while the improvements which had resulted were as traced by Mr. King. Better results

Mr. Larminie. might perhaps have been attained had the ends of the two breakwaters been aligned on a bearing about due north, the breakwaters being run due east. As constructed, their north-easterly trend was calculated to facilitate the entrance of sand from the Stockton Bight; but the extension of the north breakwater, to a point bearing due north from the end of the south breakwater, would check this tendency, and Mr. Larminie considered that the latter should not extend beyond such an alignment in respect to the north breakwater.

Mr. Lefroy. Mr. G. A. LEFROY did not see stated, in Mr. Palmer's Paper, the engineering reasons, if any, which had decided the late Mr. C. Y. O'Connor's scheme in the river as a better site for the Fremantle harbour-works than the magnificent one in Gage's Roads, the advantages of which had been recognized by competent authorities since the earliest days of the settlement in 1829. The north mole as built was in the most exposed position in which it could be placed, as regards the weather, the heavy winter gales, which did all the damage in Gage's Roads, ranging from north to north-west, after which they rapidly abated. Any other direction for the mole which inclined more to the south-west would, in his opinion, place it in a better position as regarded the prevailing heavy weather. For instance, if a single mole from Rous Head on a curved line in a south-westerly direction, and of the combined lengths of those described by Mr. Palmer, had been built instead, it would have ended on the Minden Reefs at the edge of the 5-fathom bank, and with a deep-water entrance; and the winter freshets of the river would probably have counteracted any sand-travel from the north into the south end of the roads. The scheme shown in *Fig. 6* had been suggested by him, in 1910, with a view to provide, in addition to the present inner harbour, an entrance or outer harbour of sheltered water with an area of about 400 acres, ranging in depth from 10 to 31 feet to the natural bottom. Arthur's Head (*Fig. 3, Plate 7*), being of sandstone, could have been formed into a short training-mole for the south side of the river. Mr. Palmer did not give the water-area of the harbour available for the wharfing and laying-up of ships; it seemed to be 128 acres gross, the area dredged to 30 feet at low water being less than 100 acres. Taking the figure £1,353,970, given in the Paper as the cost of the harbour described, the accommodation had cost at least £12,000 per acre. If the entrance or outer harbour-works had been laid out somewhat as Mr. Lefroy had suggested, the cost would very probably have been less than £3,000 per effective acre, and the wharves or quays would have been capable of far greater expansion than under present conditions would be possible, having regard to relative costs. Taking

into consideration the able reports and recommendations with regard Mr. Lefroy. to a harbour for the port of Fremantle, from the late Sir John Coode and from others well qualified to give advice if not so well known,

Fig. 6.



FREMANTLE HARBOUR: EXTENSION PROPOSED BY MR. LEFROY, 1910.

it might possibly be that this scheme—so costly per acre of effective water-area provided—had been decided on more through political influences than on engineering grounds. The last Annual Report

Mr. Lefroy. of the Harbour Trust Commissioners, for the year ended the 30th June, 1910, said :—

“The heavy gales experienced this winter have given the two moles a severe buffeting, and the blow of the 20th July caused a rather serious washaway on the northern face of the north mole, near the outer end. At this spot the mole is faced with heavy granite blocks, and the sea swept this protective coating completely away at one spot, and bored a hole nearly halfway across the mole. Repairs were, however, quickly taken in hand, and are well on towards completion. To provide against any serious washaway of this portion of the mole again, it has been decided to build a parapet of granite blocks along it, to give a natural protection to the clawing-down action of the seas thrown up during heavy gales. The south mole has suffered in several small spots, but these have been in no case serious, the damages being easily repaired.”

The Commissioners gave the ships which entered during the year as 672 in number with a tonnage of 1,925,548 tons, the cargo handled being 628,757 tons.

Mr. Methven. Mr. CATHCART W. METHVEN had read with great interest the Papers by Messrs. Halligan and King. They appealed specially to him, because not only had the subject with which they dealt occupied him for a very large portion of about 45 years' experience in questions affecting the improvement of the navigation of sandy estuaries and rivers, and the opening-up of heavily sand-barred rivers and lagoons in South Africa, but also the rivers of New South Wales appeared to resemble very closely in their conditions several rivers in South Africa with which he had been connected professionally during the latter 22 years of his practice. There was also a particularly close resemblance, in many respects, between the problem which had to be dealt with at Newcastle and the problem at Durban, for the solution of which he had been mainly responsible. In some cases, also, he gathered, the coastal conditions as regarded sandy beaches, and ocean and littoral currents, were also strikingly similar, though he believed he was correct in stating that while the South African coast was a rising one, the coast of New South Wales was subsiding, with the result that great natural harbours were formed, such as Sydney, Botany Bay, Jervis Bay, etc., which in South-East Africa were almost entirely absent. Mr. Halligan's Paper was none the less interesting and useful because some of the conclusions were open to criticism. Indeed, Mr. Methven was bound to say that with some of them he entirely disagreed; and he had little hesitation in attributing the failure in some of the cases to attain better results, as well as the natural maintenance of deep water at the mouths of the great lagoon harbours, to causes totally different from those assigned in the Paper. Nor could he agree with its author's conclusions as to the establish-

ment of any ratio between the area of the tidal compartment of a Mr. Methven. river or lagoon and the width of the entrance, which might serve as a safe guide in fixing the width of entrance desirable in other cases. A much safer comparison would be between the tidal volume and the sectional area, which in a certain material—sand, for instance—it might be capable of maintaining. But in that case also, owing to the variations in the material of the beds of rivers and lagoon-entrances, as well as other important conditions, it would be impossible to establish any ratio which would be of the least use in designing the proper width of entrance. The best guide, and the only one approaching reliability, was the sectional area which the tidal volume available in the particular case under consideration could maintain naturally at a point far enough within the entrance to be comparatively unaffected by the seas; and if this was ascertainable, then, at all events, it might safely be concluded that the same tidal volume would not maintain an equal sectional area nearer the sea, where it had to encounter outside forces and a continual deposition of sand. It should not be overlooked, then, that the form this deposit took would be largely controlled by the relative position of the works, as well as by outside influences; and that form might be insusceptible of removal by tidal influences, as had been the case at one period at Durban, owing to the long overlap of the southern beyond the northern work. The simple reason was that those influences, where they existed, might be dissipated by improper design of the entrance-works, as regarded width, direction, or relative length. The reference to scour as a “will-o’-the-wisp of the harbour-engineer” was one which he feared it would be very difficult to justify; for if it were not scour which had kept open some of the rivers to which Mr. Halligan referred in their primitive condition, before artificial works were resorted to at all, he must show what the force was. Mr. Methven hardly thought Mr. Halligan would deny that if there had been no inflowing and outflowing tide, the rivers except in flood, would have closed up altogether, and solid beach would have formed across the mouths. But he did agree with Mr. Halligan if he meant only to imply that the scour in some of the cases he named, on account of its comparatively small amount, could not be relied upon, however well trained by works, to keep open a navigable entrance of sufficient width and depth without the aid of dredging. This was just the point which had to be determined, in deciding whether an entrance with a navigable depth was to be created by scour alone, induced by training-works, or by scour coupled with dredging, or practically by dredging alone. It was difficult, with-

Mr. Methven. out more information than Mr. Halligan gave as to the available tidal volumes in the cases he cited, to speak with certainty. Taking, however, for instance, the Richmond River, the area of the tidal compartment was given as 6,800 acres only, and in the absence of more definite information as to the exact rise at this port, it might be assumed that the tidal range at springs was 5.5 feet. The average rise within the tidal compartment, depending of course on the condition of the entrance, and the internal regime of the river, might probably not be more than, say for argument's sake, 3 feet, in which case the volume at springs would be about 33,000,000 cubic yards. That was a wholly inadequate tidal volume to maintain anything like a good navigable depth in an entrance 1,100 feet wide, and therefore dredging on a large scale would be necessary. Yet Mr. Halligan suggested that this state of matters could be remedied by increasing the width of the entrance to 1,700 feet. In the same way he proposed to bring about the deepening of the Manning River entrance by widening it from 800 feet to 1,500 feet, and, but for considerations of internal tranquillity, he regarded 1,760 feet as a preferable width. He quoted the late Sir John Coode's recommendations in 1885, and referred to that engineer's expectation that the scour from the tidal compartment of this river would, unaided by dredging, secure a "true and uniform channel." Mr. Methven took it, however, that what was required was not only a true and uniform channel, but a channel of sufficient width and depth to take in vessels of the greatly increased draught of the present day. This was even less likely to be attained by the means proposed by Mr. Halligan than by those recommended by Sir John Coode. In these cases of rivers of comparatively small tidal capacity, the method of treatment had changed completely since 1885, with the development of the sand-pump dredger, and it was now proved beyond doubt that, while it was advisable to utilize as far as possible such means of tidal scour as might be available, so as to minimize the cost of any necessary dredging, the entrances should be designed with due regard to a proper relationship of the works to the contour of the coast in the vicinity, and to any outside currents which really mattered; they should moreover be made not wider than could be maintained by dredging to whatever depth was required; and at the same time the reduction of seas entering between the piers should also be effected wherever possible. At the port of East London, in South Africa, this had now been recognized. The existing entrance there had also been designed by Sir John Coode, and for the purpose of utilizing the tidal scour it had been originally made far too narrow to admit of the safe entry

of vessels such as had to be accommodated now. Mr. Methven Mr. Methven. therefore, recognizing that the available scour—in that case amounting only to about 1,330,000 cubic yards—was practically a negligible quantity, had advised that this difficulty should be faced squarely, and that the problem should be reduced to one purely of dredging. The works had therefore been so disposed that, while providing an entrance 600 feet in width, the northern work could readily be extended to reduce that width, should the dredging to maintain it at a proper depth be found excessive. On the other hand, at Durban Harbour, where the tidal capacity of the lagoon was about the same as at the Richmond River, namely, 33,000,000 cubic yards, the works had been designed to utilize to the utmost the whole of the available scour, while dredging had been resorted to, as recommended by Mr. Methven, to supplement it. This had been done with great success, the combined natural and artificial forces acting in one channel now giving a reliable depth of about 36 feet at low water of spring-tides, where originally the scour—untrained, and dissipated over a wide area, sometimes in several channels—could not maintain even 6 feet at low water with any regularity. Scour was a very important agent in all these cases where it was available: thus, with works still only partially constructed in 1893 at Durban, and left at that time, owing to political and other influences, in a very imperfect relationship to each other, the average low-water yearly depth on the bar had been increased by scour alone from 6 feet 1 inch in 1882 to 13 feet 8 inches in 1892; for no bar-dredging whatever had been carried out up to that date. Since then the works had been brought out level with each other, the entrance had been narrowed to 600 feet, and persistent dredging was carried on; with the result that the largest mail-steamers now entered and left the port at all states of the tide. In assisting and maintaining the work of the dredgers, the scour had been of incalculable value, and it would be folly to ignore this fact. Mr. Halligan seemed not to have discriminated sufficiently between the totally different cases of natural deep-water harbours and shallow-mouthed sand-barred rivers. There was no comparison between the physical conditions of great lagoon harbours—such as Sydney, with a natural entrance 4,900 feet in width and a depth of 90 feet, Botany Bay, with an entrance 3,467 feet wide and 60 feet deep, and Jervis Bay with one 11,590 feet wide and 120 feet deep—and rivers such as that at Newcastle, and the Richmond and Manning rivers. To argue that because a certain relationship existed between the entrance-widths and areas of these natural deep-water lagoons, therefore a similar relationship should be established at the mouths

Mr. Methven. of the sand-barred rivers, was to take up what Mr. Methven feared would be found to be an untenable position. If he was not mistaken, the coast immediately outside these lagoon entrances consisted of high headlands with deep water close inshore, where the disturbance of sand, and its transportation by means of the surf from the prevailing seas and the surf-currents set up along the shore, was infinitesimal compared with the movement which took place along ordinary sandy beaches. Mr. Halligan laid great stress on the exclusion of sand from within the entrances by making them so wide that the flood-current entering would be of less velocity than the littoral sand-bearing current. The velocity of the sand-bearing littoral currents, as he stated, seemed to range from about $1\frac{1}{4}$ to $1\frac{3}{4}$ knot per hour—or, say, from 1.4 to 2 miles per hour. It was not clear, however, whether these figures referred to the ocean-currents, properly so called, or to the eddy-currents induced by them immediately along the shores. In any case, the bottom velocities of such currents would not exceed about half this amount, or, say, 0.7 to 1 mile per hour, and would only just move sand on the bottom. The currents close inshore in shoal water were generally a good deal slower than those farther out, and their main work was to convey sand which had been placed in suspension by the surf. The actual impact, however, of a surf which was at all oblique to the coast-line—as in New South Wales it appeared to be—had a considerable effect, and the surf-current set up by it along the sandy beaches was no doubt the main transporter of sand, as it was in South Africa, where the conditions appeared to be very similar. This action of the surf, and of the waves of translation at the shallow mouths of the rivers, taking place in water highly charged with sand, was the main cause of the conveyance of sand within the entrance-works, or into the flood-current entering the river. To increase the width, as recommended by Mr. Halligan, until the flood-current between the works actually fell below the very low velocity of the littoral current outside, would merely mean the provision of an extensive trap for sand, and necessitate vast dredging-operations, which, even if successful, would entail very serious expense. The advance of the work on the travel side of the entrance beyond that on the opposite side, as also recommended, in order to ensure the littoral current carrying the sand clear of the southern wall, would be to fall into the very error which Mr. Methven had had to oppose so strenuously at Durban. Such a proceeding would only vary the form of the sand-trap within the entrance, and induce a large deposit of sand under the lee of the longer work, diverting the navigable channel and making the entrance infinitely more dangerous. Ample illustra-

tion of this from actual experience had been given in Mr. Methven's *Mr. Methven. Paper on the Harbours of South Africa.*¹

It had often appeared to him that many of these bar problems were unduly complicated by the unnecessary elaboration of investigations into wind-force, variations and velocities of currents, shifting shoals, etc., and by the many bogies sometimes raised by the layman, and, with all due respect, he might also add, occasionally by the seaman. There were in most cases certain broad principles, the consideration of which should prevail, even to the neglect sometimes of minor forces, such as sluggish littoral currents with no scouring-power and with no effect, except as regarded the transportation of sand in suspension, that would materially affect the result of the works. Even in regard to transportation of sand these currents were only aiders and abettors, as it were, of the much more powerful and effective surf-currents set up by the seas, acting perhaps obliquely, on the sandy coast. In the present case, though speaking with the diffidence that arose from want of opportunity to make personal investigations of the rivers under discussion, he thought that less theorizing on the relationship between the flood-current into the river and the ocean-currents outside, and more reliance upon the experience gained in such cases of the effects of modern dredging-plant properly applied, would speedily result in much greater improvement. Mr. Halligan gave little information as to the power of the dredging-plant in use at the harbours mentioned; but, in referring to the Richmond River, he stated that in 3 years, from June, 1906, to July, 1909, 252,040 tons of sand were removed from the bar and inner crossing by dredging, at a cost of £4,180; and he somewhat naively added, "but no improvement is reported in the depth at either place at the end of the period; in fact, rather the reverse." This quantity represented about 84,000 tons per annum only, at an annual cost of £1,393, and unless there was some error in the amounts quoted, the dredging-plant was clearly inadequate. These works at Newcastle were cited as "another example of the erroneous idea that contraction of the entrance causes a scour of the ebb-tide in a river where there is no permanent upland water." Mr. Methven was somewhat at a loss to understand Mr. Halligan's precise meaning here, because it appeared to him that the contraction of the entrance would certainly increase the velocity of the ebb on the bar, provided the entrance were not reduced so much in width as to exclude water from the tidal compartment. At Durban he had reduced the entrance-width

¹ *Minutes of Proceedings Inst. C.E., vol. clxvi, p. 31.*

Mr. Methven. from 1,100 to about 750 feet, and it had been afterwards reduced still more, to 600 feet, on the advice of Sir Charles Hartley and Sir John Wolfe Barry, for the express purpose of increasing the scour on the bar—a step which had Mr. Methven's entire approval, provided this width was considered sufficient for safe navigation. The result of this action, coupled with the dredging-operations, which were persistently carried on in the open sea whenever the weather permitted, had been entirely successful. The dredgers used at Durban were of a very powerful type. The first of them, the "Octopus," with 1,200 tons hopper-capacity, had been built to his specification by Messrs. Simons and Company, of Renfrew, and had a suction-pipe 42 inches in diameter. Others of similar type, of larger hopper-capacity, but otherwise with practically the same arrangement and pumping-power, had been supplied afterwards by the same firm. The "Cetus," the largest of these, with a hopper-capacity of 3,000 tons, usually filled her hopper in about 40 minutes, and an ordinary day's work was five to six loads, or about 15,000 to 18,000 tons per day, including depositing about 3 miles off. This was the only class of dredging-plant capable of dealing really effectively with these large bars. Economy in its use must be effected by the proper design of the entrance-works, and the resulting full use of any available natural forces. Mr. King stated in his Paper that the dredging-plant at Newcastle consisted of a dredger, "Jupiter," capable of filling a 750-ton hopper in 20 minutes, and of performing an average day's work of 3,000 tons. Here again, for such a work, the plant was obviously of insufficient power, to say nothing of the excessive width of entrance of 1,200 feet for a river with this tidal capacity, and discharging apparently across a sandy littoral. There was a discrepancy between the figure given by Mr. Halligan and that given by Mr. King as the area of the tidal compartment, the former stating it as 10,280 acres, and the latter as 6,500 acres. But, whichever it was, the width of entrance appeared to Mr. Methven to be out of all proportion, and would mean a very heavy expenditure for dredging if any material improvement in depth was to be effected. The contour-lines also, on the charts of the river-entrances which accompanied the Papers, indicated unstable conditions probably due to excessive width. Mr. Halligan, in concluding, said with reference to the Richmond and Hunter rivers: "In short, it must be admitted that, in the only two cases on this coast where contracted entrances have been tried, the expected results have not been achieved." Mr. Methven hoped that Messrs. Halligan and King would forgive him for saying that in the light of

the facts disclosed by their Papers, there was no cause for surprise Mr. Methven. at this, even although, judging from the original condition of the Hunter River at Newcastle, as shown by the charts of 1816 and 1866, the conditions before the works were commenced did not appear to be so severe as they were on the east coast of South Africa.

Mr. W. T. OLIVE was glad to see that the effect of the river at Mr. Olive. Newcastle in time of high flood was recognized as being a scouring action; thus on p. 150 it was stated that large quantities of sand were swept well out to sea and dispersed in deep water. He could not see how, with such large catchment-areas—in this case 8,000 square miles, compared with 10 square miles of tidal basin—it could be otherwise. Mr. Palmer, in describing the West Australian rivers, stated on p. 159 that the volume of their waters was so small that the rivers were unable to do more than occasionally clear their mouths of extraneous sand; while on p. 162 he said that the northernmost of the lakes discharged into the sea, the opening being maintained by the River Murray, which in heavy freshets cut through the sand bar at Mandurah. Whilst it was not possible to generalize, it appeared to Mr. Olive that where rivers flowed into harbours having narrow entrances the ideal of the harbour-engineer should be to head up the river during the flow of the tide and release it on the ebb. If that could be accomplished by sluicing-basins or otherwise, the full benefit of the land-water would be realized; in other words, whatever would conduce to a greater velocity during the ebb-tide than during the flood-tide was beneficial to the maintenance of depth and the avoidance of dredging. He was quite in accord with Mr. Halligan's remark that each case must be treated entirely on its merits, and therefore disagreed with his suggestion, for the design of future harbours, of endeavouring to establish a fixed ratio of harbour-entrance to tidal compartment. The Paper was valuable, inasmuch as it did not scruple to point out errors in the past. Mr. Olive might draw attention to the new proposal for extension at Cape Town, South Africa, which was, to impound a water-space of 431 acres by a mole 8,550 feet long, with a single opening of 350 feet. This space received the washings of a large town (all the storm-water drains delivering into it), despite the fact that a large dock of 12 acres, excavated in rock, had to be abandoned some years ago, after great expense, for the sanitary reason that there was no means of changing the water. It was hoped, however, that this would be effected occasionally by flood-water, as was the case at Curl Curl and Narrabeen lagoons, mentioned on p. 129.

Mr. Price. Mr. JAMES PRICE, while considering that Mr. Halligan's Paper gave an interesting summary of facts relating to natural and artificial harbours in New South Wales, and to sand-movements along the coast, thought the conclusions drawn from these facts were hardly in accordance with the views of engineers who were concerned with estuary harbours in the United Kingdom. In the former country it appeared that the main factor in sand-movement on the coast was the ocean-current from north to south—assisted or impeded by the surface currents set up by the wind acting on the surface of the ocean—the tidal currents being negligible, as they set straight on shore. From this it was concluded that the ocean-current carried a quantity of sand, which it conveyed into a harbour when the tidal current at the entrance flowed at a greater velocity than that of the ocean-current. Why should this be? The quantity of water flowing into a harbour each tide was the tidal volume of the harbour less the inflow of river-water for the period of rise of tide; and the sand bearing a certain ratio to the sea-water, a fixed quantity of sand would be brought in each tide, independently of the velocity of the current at the entrance. Again, the quantity of water running out at ebb-tide would be the tidal volume plus the river-water; and provided the entrance-channels were sufficiently narrow to make the velocity such as would lift the sand deposited on the bottom, the harbour would be self-cleansing. If Mr. Halligan's theories were correct, harbours with natural entrances should have their tidal areas reduced by reclamation, so as to reduce the velocity of the currents at the entrance to less than that of the ocean-currents; and artificial harbours would be constructed with wide entrances, for the same reason. It seemed more natural to assume that the ocean-currents were charged with sand picked up near the shore by wave-action, the part of the current nearer in being the most heavily charged; then converging training-walls run out to sea caused the flood-tide supply to be taken from deeper and cleaner water, and prevented the sand and gravel of adjoining beaches from entering the harbour. The comparison of entrance-width with tidal area could not lead to any very definite results; it would be necessary to take the depth into consideration and compare the sectional area of the entrance with the tidal volume of the harbour. In order to arrive at a correct estimate of the river-water, the volume and duration of winter floods should be considered, rather than the catchment-area. The Table on p. 257 gave approximately the conditions affecting the currents at eight different places in Cork Harbour at half-tide, with an ordinary winter flood. The first four sections were of artificial channels; of these three required

Mr. Price.

CORK HARBOUR CHANNELS.

	1	2	3	4	5	6	7	8
Distance from Cork Miles	0	1	2½	3½	7	11	13½	15
Width Feet	200	300	360	360	1,000	3,500	3,000	5,000
Maximum depth "	26	23	24	24	59	37	100	74
Sectional area Square feet	5,200	6,200	7,700	8,400	45,000	110,000	175,000	210,000
Flood Cubic ft. per sec.	300,000	330,000	340,000	376,000	403,000	418,000	568,000	570,000
Tidal area Acres	44	126	440	570	4,500	5,600	13,400	14,000
" volume . Millions of cubic yards	0·8	2	7	9	55	86	212	221
Mean current from flood . Ft. per sec.	57·7	53·2	44·1	44·8	8·95	3·8	3·24	2·72
" " tidal volume "	12·3	16·8	30·9	50·2	166·05	96·2	136·76	117·28
" " " " " "	70	70	75	95	175	100	140	120
" " " " " Knots per hour	0·69	0·69	0·74	0·935	1·72	0·985	1·38	1·18
Surface current "	1	1	1½	1½	2½	1½	2	2
Siltage per annum Inches	3	8	8	0	0	0	0	0

Mr. Price. maintenance-dredging, the fourth was maintained by the natural scour of river and tidal water. The last four were natural channels maintained chiefly by tidal currents, the last being at the harbour's mouth. It would be observed that in the upper channels river-water was the more important, while in the lower channels tidal water performed most of the work of cleansing. Thus the river-water acted indirectly on the entrance-channel by keeping the upper reaches clear to admit tidal water. If Mr. Halligan's theories were correct, Cork Harbour ought to have silted up long ago. The harbour faced the prevailing wind, the ocean- and flood-tide currents made towards the mouth at a velocity of about 1 knot per hour on the surface, and the velocity of the current in the entrance-channel was 2 knots. In place of silting, the depths were improving in the natural channels, and only about half of the artificial channel required maintenance-dredging. The velocities of the flood- and ebb-tide currents were about the same, but the duration of the ebb-current was about 7 hours to $5\frac{1}{2}$ hours of flood-current. The velocity of the ebb-current within $\frac{1}{2}$ hour of low water was sufficient to lift small gravel from the bottom and in 36 feet of water in the entrance-channel. This sea-gravel appeared only in small quantities about 1 mile inside the mouth. The river-water brought down mud and peaty matter, part of which was deposited when the sectional area of the channel became too large for the quantity of river- and tidal water available.

Mr. Rossbach

Mr. WILLIAM ROSSBACH considered that Mr. Halligan had brought together a large amount of information as to ocean- and river-currents, winds, tides, etc., which would be of considerable value to the engineers dealing with the rivers of New South Wales. Should, however, some of his deductions be correct, the position in regard to the partially-constructed harbours there became very serious, as would be manifest from an inspection of the accompanying Table, showing the widths between the breakwaters of the various rivers as designed, and as they should be, according to Mr. Halligan's formula of 1 foot width to each 3 acres of tidal compartment. Especially was the matter serious in view of the Works Department having expended about a million and a half sterling on these partially-constructed entrance-works: but in Mr. Rossbach's opinion the great differences existing between the conditions of the various rivers had not been recognized sufficiently by Mr. Halligan, and as a result some of his deductions were unwarranted. The following particulars would show how varied were the conditions. The true rivers, which did not fall into wide estuaries but came abruptly on to the coast, were the Tweed, Richmond, Clarence, Bellinger, Nambucca, Macleay, Hastings, Manning, Hunter (Newcastle), Crookhaven and Moruya.

Mr. Rossbach.

River.	Author of Design being carried out.	Width of Entrance as designed.	Width by Mr. Halligan's Formula.
		Feet.	Feet.
Tweed	Mr. C. W. Darley . .	500	1,667
Richmond	} Sir John Coode . . {	1,000	2,267
Clarence		1,400	11,333
Bellinger	} Mr. C. W. Darley . . {	500	547
Nambucca		500	910
Macleay	{ Sir John Coode . . . { Mr. C. W. Darley . . }	700	1,250
Hastings		650	2,133
Camden Haven	} Mr. C. W. Darley . . {	400 ¹	2,413
Manning		800	2,267
Port Forster	Mr. C. W. Darley . .	400	7,313
Newcastle Harbour	{ Mr. E. O. Moriarty and Mr. C. W. Darley . }	1,200 ²	3,428
Lake Macquarie		1,300 ³	9,300
Crookhaven	Mr. T. W. Keele . .	600 ⁴	3,110
Moruya	Mr. E. O. Moriarty . .	950 ⁵	1,620

¹ Curved northern breakwater added by Mr. T. W. Keele.

² Entrance diverges slightly seaward : minimum width, 1,200 feet ; maximum, 1,450 feet.

³ Reduction to 200 feet recommended by Sir John Coode.

⁴ Widened to 1,300 feet by Mr. E. M. de Burgh.

⁵ Inside work (training-wall) proposed by Mr. Darley, practically reduces the width to 550 feet.

Camden Haven was a small inlet having at its back some lakes of considerable area, in which there was only a few inches of tidal range. Cape Hawke harbour (or Port Forster) was somewhat similar. The entrance to Lake Macquarie was by a narrow channel about $3\frac{1}{2}$ miles long, extending from the ocean to the lake, which had a water-area of 27,900 acres, and was practically tideless. Bateman's Bay was quite different from all of these, being about 5 miles wide at the coast-line, and narrowing to about 900 feet 5 miles back from the coast, where it received the waters of the Clyde River. The bay had a rocky shore on both sides. Wagonga was an inlet of the sea, carrying water 40 feet to 50 feet deep over the larger portion of its area, and was generally similar on a small scale to Sydney Harbour, having a rocky coast on both sides of the entrance. The Manning had two entrances, one at Harrington and the other at Farquhar, 7 sea-miles distant, both of which admitted large quantities of tidal water and allowed of the discharge of flood-

Mr. Rossbach. waters. Crookhaven, with a drainage-area of only 40 square miles, was connected with the Shoalhaven (drainage-area about 2,761 square miles) by what was known locally as "The Canal," which carried a large portion of the Shoalhaven waters to and from the Crookhaven entrance. The Bellinger, Macleay (new entrance) and Manning rivers had no rocky headland on either side, but only sandy beaches, extending in some cases for several miles. The remaining rivers had a headland on one side and a sandy beach on the other. The length of these beaches varied, some of them being many miles in extent. The small places, Tuggerah, Lake Illawarra, Tuross and Bermagui were frequently closed against the sea. It should also be pointed out that the average range of tide in the different tidal compartments varied very considerably, as also did the mean distance of the compartment from the ocean—both matters of great importance. There were also differences in the rainfall, steepness of the catchment-area, nature of the soil, etc., as well as in the extent of the improvement-works carried out. In some cases the works were well advanced; in others, less than half the work proposed had been carried out; while in a few cases no works had been undertaken. Notwithstanding the very varied conditions of these rivers, Mr. Halligan, from a consideration of some of the conditions at three of them, had devised a formula which he would apply to all the rivers on the coast. There was no analogy between Sydney Harbour, Botany Bay and Jervis Bay, on the one hand, and the various rivers on the other. The places mentioned were not rivers, but inlets of the sea; none of their entrances had a sandy beach on one side or the other, but bold precipitous headlands with deep water close up to the shoreline; while they all had a rocky coast ranging in length from $1\frac{1}{2}$ to 8 miles on the one side and from 4 miles to 10 miles on the other. In other words, there was an absence, close to these entrances, of the sand which, as Mr. Halligan said, was "put into suspension by the waves, and slowly moving with the current along the beach, at length reaches an inlet, and is carried into the estuary by the flood-tide." The beaches in these cases were miles away. As far as observations went, the larger portion of the littoral drift on this coast occurred over a strip extending from high-water mark to 2 or 3 fathoms below low water; but if any sand got into the three places named, it must be at depths of 60, 90 and 120 feet below low water, where there was no wave-action such as that which placed sand in suspension on the beaches. Again, the three tidal compartments referred to had each a tidal range equal to that of the ocean outside, whereas in the case of every river mentioned by Mr. Halligan the range was only about one-half of that

outside. For instance, the Clarence River, which had the largest Mr. Rossbach. tidal compartment, though subject to tidal influence over about 68 miles of its length, had a range at its mouth of about 5 feet at spring-tides, and at 20 miles up-stream only about 1 foot 6 inches. The Hunter River had $5\frac{1}{2}$ feet at the entrance, and at Maitland, about 40 miles up-stream, only $2\frac{1}{2}$ feet. In some cases, such as at Camden Haven and Port Forster, the average range was probably much less than in the typical cases mentioned, owing to a large portion of what was called "tidal compartment" consisting of lakes wherein the tidal range was only a few inches. Many of the tidal compartments were also incapable of being completely filled and emptied by tidal water, no matter what the entrance-width might be. It was obvious that a formula based on a full range of tide in the tidal compartment could not apply to a compartment where the possible range was only one-half or less. That the factor was not the area of the tidal compartment but its capacity for tidal water would be clear when the areas of tidal compartments of those rivers having somewhat similar conditions were placed in order of their size, and their depths were compared. It would be found that they fell into practically the same order, namely:—

Rivers at which breakwaters had been built on both sides of the entrance—

	Tidal Compartment (Approximate only). Acres.	Depth on Bar at L.W.S.T. Feet. Inches.
Hunter (Newcastle)	10,280	22 0
Richmond	6,800	10 3
Moruya	1,620	7 2

Rivers at which no breakwaters had been built, or only on one side, and that the rocky side—

	Tidal Compartment (Approximate only). Acres.	Depth on Bar at L.W.S.T. Feet. Inches.
Clarence	34,000	11 4
Crookhaven and Shoalhaven	9,330	11 3
Macleay	6,800	7 10
Hastings	6,400	6 6
Tweed	5,000	6 5
Clyde (Bateman's Bay)	4,253	5 10
Manning	3,750	7 4
Nambucca	2,730	5 4
Bellinger	1,640	5 5

Rivers which had tidal compartments consisting largely of lake areas—

	Tidal Compartment (Approximate only). Acres.	Depth on Bar at L.W.S.T. Feet. Inches.
Lake Macquarie	27,900	4 3
Cape Hawke	21,938	3 10
Camden Haven	7,240	3 4

Mr. Rossbach. Wagonga Inlet, with an area of 1,600 acres, and a depth of 10 feet 4 inches, stood alone, in that it had an excellent tidal compartment in which the tidal range was more nearly equal to that of the ocean than any river in this list. Mr. Halligan could find only two causes why the three harbours on which he based his formula did not shoal more, namely, the projection of the north headlands and the wide entrances. Mr. Rossbach would suggest a third—that it was mainly due to the heaping-up action of onshore gales producing an under-current seaward, which removed periodically any sand that might find an entrance. The waters of Sydney Harbour were known to have been raised 10 inches above the ordinary tidal level by this means, and it was certain that Port Kembla Bay and other similar bays on the coast had been so cleaned out periodically.

Another conclusion at which Mr. Halligan arrived was that there was no relation between the area of a catchment and the depth on the bar. Mr. Rossbach was of opinion that though the tidal waters were the main factor in determining the depth on the bar, the upland waters were of very considerable value in strengthening the ebb-tide. In the case of the rivers of New South Wales, where there was an absence of wide estuaries, where the rivers came abruptly on to the coast, and where the tidal range was small, the upland water was not the insignificant factor that it might be in the case, say, of rivers on the English coast, where the range of tide might be as much as 27 or 28 feet, and the ratio of fresh to tidal water was only 1 to 250, 1 to 300 or even 1 to 1,500. For instance, taking the case of the Hunter River—with a catchment-area of 8,269 square miles, an average annual rainfall of 36 inches, and a tidal compartment of 10,280 acres, with an average range of 3 feet 6 inches—and assuming the low figure of 30 per cent. for discharge, the average quantity of upland water discharged per annum would be equal to between one-fourth and one-fifth of the quantity of tidal water entering the river in the year. A large portion of this upland water was discharged in freshets and occasional floods, when it scoured out sand that had accumulated during periods of small flow; but there was always more or less water coming down the river which, ponded up during the flood-tide, was discharged on the ebb. This was thereby more or less prolonged, and he considered this extension of the period of the ebb to be a factor of some importance in maintaining the depth over the bar in the case of New South Wales rivers. Mr. Halligan pointed out that only the same quantity of tidal water passed out of a river as previously entered; that the velocities which he had measured on the ebb and flood at any one place were equal; and that the flood-tide was assisted by the waves in bringing sand into the entrance: but as there were no waves to disturb the sand on the

ebb, the inference to be drawn was that the upward forces through the entrance were greater than the downward. It was obvious, however, that there must be some equal force or forces to carry the sand out again, otherwise these rivers would ages ago have ceased to exist. As an instance of the enormous quantity of sand which was sometimes scoured out by upland water in time of flood, the Clarence River might be mentioned. Here, in the flood of 1887, the unprotected portion of the northern spit was swept entirely away, leaving 33 feet where previously the spit had been dry at low water; and 32 to 40 feet on the bar, where previously there had been about 12 feet. It was difficult, therefore, to see no relation between the area of the catchment and the depth on the bar. It was interesting to note that while the tidal water was the main factor in maintaining the depth on the bar, the relation of the catchment-area to the depth was also well marked when the catchment-areas were placed in order of their size:—

Mr. Rossbach.

Rivers at which breakwaters had been built on both sides—

	Catchment-Area. Square Miles.	Depth on Bar at L.W.S.T.	
		Feet.	Inches.
Hunter.	8,269	22	0
Richmond	2,683	10	3
Moruya.	609	7	2
Lake Macquarie	291	4	3

Rivers at which no breakwaters had been built, or only on one side, and that the rocky side—

	Catchment-Area. Square Miles.	Depth on Bar at L.W.S.T.	
		Feet.	Inches.
Clarence	8,505	11	4
Macleay	4,581	7	10
Manning	3,164	7	4
Crookhaven and Shoalhaven.	2,801	11	3
Hastings	1,389	6	6
Bateman's Bay	696	5	10
Nambucca	552	5	4
Cape Hawke Harbour	514	3	10
Bellinger ¹	479	5	5
Tweed ¹	418	6	5
Camden Haven	238	3	4

When it was remembered that large lake areas existed at Lake Macquarie, Cape Hawke, and Camden Haven, which acted as balance-reservoirs and retarded the flow of the upland water, it appeared more than a coincidence that the depths on the bars fell into place so well. In order to prove that there was no relation

¹ Breakwater works further advanced than in the case of Nambucca.

Mr. Rossbach. between the catchment-area and the depth on the bar, Mr. Halligan instanced the Clarence River, with 8,505 square miles, and Crookhaven, with only 40 square miles, having equal depth; also the Richmond, with 2,683 square miles, having the same depth as Wagonga, with only 52 square miles. With regard to the first comparison, it had already been explained that the Crookhaven carried, in addition to its own waters, a large portion—and sometimes the whole—of the Shoalhaven. The explanation of the second case was that at Wagonga, with its excellent tidal compartment, the value of the tidal water far outweighed that of the upland water. This tidal compartment was very spacious, was close to the ocean, had a large tidal range—at spring-tides 4 feet 6 inches at the entrance and 3 feet 6 inches at its extreme distance from the ocean—and was rapidly filled and emptied each tide. It was certain that, compared with other rivers having similar areas of tidal compartment, much more tidal water flowed into Wagonga; hence the better scour. In passing, the inference that might be drawn that the Clarence River entrance, after having had £466,000 spent on it, was to be regarded as a failure, should be corrected. Of the amount mentioned, £182,830 had been spent on a scheme subsequently abandoned in favour of one proposed by the late Sir John Coode, the expenditure on which had been £283,419. The first work recommended by Sir John Coode was the building of the first sections of the breakwaters, the inside training-walls to follow. On the recommendation of a non-professional Parliamentary Committee, however, all the internal works had been constructed, but the outer or breakwater works, which, if built, might have been expected to improve the bar and prevent the ingress of sand, still remained untouched, as did also the removal of the rocky patch in the entrance, which Sir John Coode proposed to remove to a depth of 18 feet at low water. Had the works been carried out in the order recommended, the results their designer anticipated would, in Mr. Rossbach's opinion, have been fully realized. Mr. Halligan considered that the only effect of £422,000 spent on the harbour scheme at the Richmond River had been to fix the bar, which formerly wandered about over 5,000 feet of coast. He overlooked or had forgotten, however, the difficulties and dangers encountered by vessels before these works were commenced; also the fact that, as the improvements progressed, the shipping-companies increased the size of their vessels, that these now entered practically at all times, and that they were seldom delayed except by bad weather. Sir John Coode had anticipated a depth of 12 feet at low water, and for many

months together the depth ranged from 10 to 15 feet, though Mr. Rossbach. the scheme had not yet been completed, and there had been an almost entire absence of flood conditions for 17 or 18 years. The sandbank referred to as encroaching on the channel was due to a combination of causes. It lay on the convex side of the river, where there would be a natural tendency to shoal; also, large quantities of sand were blown over the breakwater during the prevalence of southerly winds, and were trapped on the shoal ground, over which the velocity of the tide was very low and not sufficient to move the sand under ordinary conditions. This Mr. Halligan was now endeavouring to stop by the planting of marram-grass and other vegetation. Again, it was stated that the sand was fast filling the estuary. This was not correct: as the training-works progressed, the sand which had accumulated in the estuary moved downstream, as shown by periodical soundings. A large portion of the sand-spit shown in Fig. 7, Plate 5, had disappeared, and the recent reports stated that the river-entrance was in better condition than ever before. True, owing to recent heavy weather the entrance had shoaled to 8 feet, and assistance was now being rendered by dredging; but this was only temporary and was liable to happen no matter of what width the entrance might be. The remark about a large amount of dredging having been carried out, both at the Richmond River and at Newcastle, without any resulting increase in depth on the bar, was misleading. The larger portion of the dredging at the Richmond River during recent years had been done not for the purpose of deepening the bar, but for widening the channel at the upper end of the sandbank shown on the plan, where there was a very sharp turn, difficult to navigate. At Newcastle the dredging was not at the point where the least depth was reported by the pilot (this being rocky and not capable of being dredged), but to the north of the fairway-line, where there was now better water than previously, as the result of dredging. This explained how it was that, as Mr. Halligan said (p. 146), no improvement was reported in the depth at either place at the end of the period.

The views expressed as to the effect on the ocean-current of the various-shaped headlands, in Mr. Rossbach's opinion, needed to be supported by many more observations than had been made before they could be fully accepted. An inspection of the coast-charts showed no headland concave to the north which would cause the counter-current said to set to the north past the Richmond River entrance; nor did it appear probable that Crowdy Head caused a set to the north past Harrington and Farquhar Inlets, $3\frac{1}{2}$ and $10\frac{1}{2}$ sea-miles distant respectively from the head. Other similar cases might be

Mr. Rossbach. mentioned. It was also difficult to see how, if the north-east wind increased the velocity of the counter-currents, the southerly winds should also accelerate them, as stated by Mr. Halligan. With regard to littoral drift, his own observations had led him to the conclusion that the greater part of the sand moved over the space between high-water mark and the 12- or 18-foot contour-line below low water. The sand was put into suspension by the waves and carried by the alongshore current, which might be accelerated, retarded, or reversed by the winds, according to their direction and force. Strong winds in some places caused the waves to break with a certain amount of obliquity on the shore; this also moved the sand in the direction of the wind. The velocities of the currents measured close alongshore, where most of the drift took place, were very low (generally about $\frac{1}{4}$ knot per hour or less), and they were easily reversed by the wind; consequently, notwithstanding that the ocean-current ran to the south, large quantities of sand could be moved to the north by the southerly gales, which were much more numerous than those from the north. He thought Mr. Halligan undervalued very much the effect of the southerly gales on the movement of sand; in some cases it could only be described as enormous. He had seen the southern end of a beach denuded of sand by a south-easterly gale to such an extent that, during 8 to 9 months of normal weather afterwards it was not made up again to its previous condition. On the coast of New South Wales, where the prevailing wind came from the north-east, not only did the river-entrances at the southern end of beaches shoal, as stated by Mr. Halligan, but those at the northern end shoaled also. The position as regarded sand-movement on the coast appeared to be a struggle between the ocean-current acting almost continuously from the north and carrying sand in comparatively small quantities, and occasional heavy gales from the south during which the sand was disturbed to a much greater depth, and carried in very large quantities to the north. Whether the ultimate movement of the sand was from north to south, or the reverse, there was not sufficient evidence at present to show.

Mr. Halligan considered that the width of entrance should be so determined that the velocity of the ocean-current should be greater than the flood-tide between the ends of the breakwaters, in which case the sand would be carried past in the stronger stream and would be harmless: and he stated that if this condition could be obtained, all dredging within the estuary became a permanent improvement. With the low-velocity coastal current, as measured off a number of the river-mouths, it would appear to

be impossible to do this without making the entrance so wide ~~Mr. Rossbach.~~ that the most permanent thing would be the dredging. The tidal compartment had to be filled with water from the ocean; and if such water carried sand put into suspension by the waves, some sand must necessarily pass into the entrance, whether by a low- or a high-velocity current. The low-velocity flood-current due to an excessively wide entrance would admit of the precipitation of the sand close to the river-mouth. On the ebb-tide, however, the precipitated sand would not be disturbed to the same extent by the waves, and this low-velocity current would be powerless to move it seaward: hence an accumulation, which would require very frequent dredging to maintain a navigable channel through it. While Mr. Rossbach—who held the position of Principal Assistant Engineer in the Harbours and Rivers Branch of the Public Works Department, and who had had 34 years' experience in connection with harbour-works and for about 23 years had done all the designing of the works described by Mr. Halligan, under the various Chief Engineers—held views at variance with Mr. Halligan on some points, he desired to express his appreciation of the effort made to throw light on one of the most difficult problems in engineering.

Mr. W. H. SHIELDS was much interested by Mr. Palmer's Paper ~~Mr. Shields.~~ on Fremantle Harbour, which although brief contained so much information that little remained to be added, except concerning the various graving-dock proposals. He thought, however, that the author would be interested to know that since he severed his connection with the Department and State which he had graced as Engineer-in-Chief, the two small patches of silting had practically disappeared, the whole harbour showing signs of scouring out. There were now few places with less than 33 feet at low water (phenomenal low water, only once known). Were it not for the large quantity of stone thrown in to protect the railway-bridges from scour, probably the whole of the river, with the exception of the rocky bar, would scour out to 40 feet or deeper between the entrance and Melville Water. The two small patches of silting were generally supposed to be due to eddies formed by the filling originally left below the mail-boat jetty. The electric cranes had proved highly successful in working.

Mr. VOISIN, of Boulogne-sur-Mer, remarked that the history of ~~Mr. Voisin.~~ the development of the approach to the Newcastle Harbour possessed the greater interest because it so rarely happened that engineers were entrusted with the carrying out of like work where research could furnish such useful information and such valuable instruction. Generally speaking, Mr. King showed that the state

Mr. Voisin. of affairs, which was risky and even dangerous in 1816, had been made, if not perfect, at all events entirely satisfactory, by the combined influence of structural works and increasingly extensive dredging; and he pointed out that the proposed new extensions, coupled with suitable dredging, rendered it probable that the results hoped for—namely, a channel with a depth of 30 feet at low water and a reasonably safe approach in all weathers—would be achieved. Mr. Voisin concurred in this hope, as being in conformity with the accepted data upon the subject. It would indeed be difficult to claim that a similar result could be obtained exclusively from the construction of piers by making the opening between them more or less equal to that suggested by Mr. Halligan, which in this case would be 2,570 feet in place of 1,200. It seemed desirable, in Mr. Voisin's opinion, to draw attention to a want of concurrence in the two Papers, the wetted area being given in Mr. King's Paper as 6,500 acres, and in Mr. Halligan's Paper as 10,280 acres, while the width between the breakwaters, according to the former (scaled from the plan), was in 1907 about 1,450 feet, as against 1,200 feet given in the latter. It was noticeable that the distance between the heads, a distance which Mr. Halligan regarded as a basis of design, had been gradually reduced from 2,750 feet in 1903 to 2,000 in 1907, while the effect of the new works proposed would be to reduce it to 1,900; but that, notwithstanding this, the position, instead of being made worse, had been very appreciably improved, thanks, in a measure, to dredging. Mr. King gave the current during floods as having a velocity of $7\frac{1}{2}$ knots per hour, and, during the ebb, 2·6 to 1·7 knots (Chart of 1909), in place of the 3·3 knots stated in Mr. Halligan's Paper. This velocity had diminished progressively, and would doubtless continue to do so. Could it be said that the extension of the north breakwater would stop for an indefinite time the invasion of the channel by sand coming from Stockton Bight? Mr. Voisin did not think so, and he agreed with Mr. Halligan that the effect would be only temporary, and that it would without doubt be necessary to resort to further extension, until a point was reached where the depth of water was sufficient to prevent further sea-action on the sand—a point which would be reached the sooner by reason of the low velocity of the littoral current. However, the fact must not be lost sight of that dredging would permit any given state of affairs to be maintained during a longer or shorter period, according to the extent to which it was employed—an extent which would appear to be somewhat large owing to the considerable movement of the sand concerned, but which was amply justified by the magnitude of the interests

involved, and especially by the economic development of Newcastle. Mr. Voisin. The colossal work carried out with success in the Mersey Bay showed what could be done in a proportionate way at the mouth of the Hunter River.

Too much stress could not be laid on the remark made by Mr. Halligan at the beginning of his Paper, that each harbour constituted a special case, and it was thus necessary, in designing any works, to apply the general principles after having thoroughly grasped the local circumstances. It was one of these general principles which Mr. Halligan endeavoured to establish for the comparatively simple case where no tidal current—at any rate parallel to the shore—existed, and the littoral current (ocean-current) ran in a constant direction. As stated in the Paper, it was more usual for the littoral current to be absent, or if present to be ill-defined, while it was the tidal current setting along the shore which predominated and undoubtedly caused complications in the problem. It seemed possible, in passing, to sum up the principles laid down more or less broadly by different engineers (Messrs. Coode, Haupt, and others), by observing that the movement of sand was the result of a variety of forces—currents of all kinds (littoral, tidal, or fresh-water), winds, etc.; and naturally the larger the number of these acting forces the more intricate was the examination of the phenomena: and thus in the special case of the New South Wales coast where there was neither an alongshore current nor any appreciable upland water, it had seemed possible to Mr. Halligan to formulate a law, the accuracy of which it would be interesting to test in other similar situations. It was according to the general law indicated above that in cases similar to that on the New Jersey coast the movement of sand was dependent much more upon the tidal and littoral currents than upon the wind, the mean effect of which did not always coincide with the tidal current, although very often it assumed the direction of the flood. Mr. Halligan analysed what had taken place at the entrance to one of the estuaries in New South Wales. The observations appeared reasonable, but possibly it was undesirable to adhere too closely to these deductions, which were dependent upon a determination to avoid sanding up—a determination which was certainly reasonable, but which should not be considered to the exclusion of others which might be of even more importance: for frequently, even in the case of Newcastle Harbour, at the mouth of the Hunter River, it was neither very difficult nor very costly to successfully combat the accumulation of sand. Thus, Mr. Halligan recommended that the breakwater which opposed directly the littoral current should be

Mr. Voisin. lengthened beyond the other, and it might easily happen in certain cases that a like arrangement could be avoided, for the convenience of navigation, only by having recourse to a little dredging. In the same way he recognized with justice that, in order to obtain greater stillness in the harbour, it might be necessary to reduce the entrance-width below the limit which he indicated. He pointed out that the width at the entrance which he contemplated was the distance between the ends of the breakwaters, and not necessarily the shorter distance across the entrance. It seemed difficult, however, especially if the line joining the heads was very oblique to the centre-line of the channel, as, for instance, in the design proposed by him for the improvement of the entrance to the Richmond River, to admit that the width measured upon this line would indicate the same as that measured at right angles to the centre-line of the channel. Besides, in the three examples which had served to illustrate the proposition, the two lines were practically coincident, a fact that did not warrant the assumption that these two widths were equivalent. Mr. Voisin would like to add

(1) That the formula brought forward for the width of an entrance, namely, 1 foot per 4 acres, was applicable only to large estuaries like those cited; for such as were of less importance it would involve widths very much too small, so that, when considering areas of less than 1,000 or 1,200 acres, the exigencies of navigation must be the sole guide.

(2) That the formula itself seemed capable of application only with the greatest reserve, as it ignored a number of important factors, such as the duration of the tide, the section of the channel, etc. It also seemed inadmissible to assume that the flood-tide should have a greater velocity than the littoral current in order to bring into the estuary only a slight deposit. Further, a roughly approximate calculation seemed to show that the proportion of 1 foot to 4 acres was rather on the small side than otherwise. However, Mr. Voisin thought that conclusions such as those laid down by Mr. Halligan, when relieved of numerical significance, were of considerable interest.

With regard to the Paper by Mr. Palmer, in the absence of plans of the harbours of Albany and Banbury, it was difficult to form any definite opinion upon the very brief description of those works, but the general situation on the west and south coasts of Australia presented some similarity with that on the east coast. Thus, it was noticeable that a regular current existed, almost constant in direction (except during gales from the north-west, which appeared to be

infrequent) and somewhat similar to that on the east coast, but Mr. Voisin. Mr. Palmer made no mention of the direction taken by any tidal currents in the open sea. It would have been interesting to know if, as on the coast of New South Wales, these approached at right angles to the shore-line, and were thus without any appreciable influence on the nature of the bed in the open sea or in the estuaries. No doubt this lack of precision was due to the insufficiency of the observations made on this great length of coast; but failing further information on this head, it would be interesting to know whether the relation found by Mr. Halligan to exist, between the width at the mouth of an estuary and the wetted area of such estuary, obtained on the western and southern coasts of Australia, as well as upon the eastern, since it seemed that in the former also the action of the land water might be considered as of small importance. As regarded Fremantle Harbour, it was easily realized that the engineers were concerned on account of the sand-movement at its entrance, especially at a time when the manufacture of dredging-plant was in its infancy. It was also apparent that these fears must have been the more real because at that time the financial resources of the Colony were of the smallest, and it was difficult to estimate what the future held in store. The history of the design and execution of the works at Fremantle, as related by Mr. Palmer, placed also in evidence the hesitancy and mistakes to which the incomplete knowledge of the situation had led; but in a new country, as Western Australia then was, perhaps it was better to run a risk than, by hesitating, to entail a delay in the economic development of the country; and boldness was doubtless more to be desired than extreme prudence. The solution eventually adopted seemed certainly the best, for it was preferable to locate the new berthage immediately within the mouth of the Swan River and in the town itself, rather than at a distance as proposed by Sir John Coode. As Mr. Palmer did not give the weight of the large stones tipped on the outer slopes of the breakwaters, especially of the north one, it was impossible to criticize the section adopted. However, on referring to Figs. 6, Plate 7, and to the description in the text, it was evident that even during the north-westerly gales the sea was not heavy, a state of affairs which might to some extent be expected from the lie of Fremantle. The somewhat crude system employed in executing the rock facing under the quays could not give entirely satisfactory results, even when the material was as suitable as that at Fremantle, but it was evident that the settlement stated to have taken place, especially at the south quay, had not been attended with any serious consequences. Some more detailed account of the jarrah timber

Mr. Voisin. employed in the construction of the quays would not have been without interest, though already it was known that Australian jarrah was eminently capable of withstanding the attack of the teredo and of marine worms. Fig. 9, Plate 7, did not show how the electric cranes would be installed, as on the quay-side two railway-tracks only were shown and no crane-road: it must, however, be assumed that the width of the quay was sufficient for the cranes to be placed astride the other tracks. An extremely interesting point was the wind-screen placed along the shore, immediately behind the north quay; but from its construction, as described in the Paper, it seemed again to be clearly indicated that the sea was not very heavy at this point. With regard to the arrangement of light-sectors at Woodman's Point, it was curious to note that red faced the shallower navigation while green was used to mark the dangers; it would have been more in conformity with tradition if the reverse had obtained. It was to be regretted that the cost of dredging by bucket dredgers had not been given, as had been done in the case of sand-pumps, where the figure was almost as high as might be expected for good bucket dredgers. In conclusion, it was interesting to note the success of the works undertaken at Fremantle, a success which did credit to the engineers responsible for their design and for their subsequent construction.

Mr. Walmisley. Mr. A. T. WALMISLEY thought that, while Mr. Halligan emphasized the importance of adequate width of entrance, he appeared to overlook the value of depth, which was a controlling element in the discharge of a channel. In considering a wide entrance it must be remembered that, when the water within the entrance became slack, it deposited any sediment held in suspension; unless, therefore, a scour was created to maintain a tidal channel of ample capacity for the requirements of the harbour, the cost of dredging became a constant item. Experience alone would enable the harbour-engineer to decide the mean between the advantages of a width sufficiently narrow to maintain scour and sufficiently wide to provide facility of entrance at all states of the tide.

Mr. Walsh. Mr. H. D. WALSH, having been Government Engineer in charge of the Newcastle harbour-works for a number of years previous to 1901, was much interested in Mr. King's Paper. During the earlier stages, from 1866 to 1880, owing to want of funds, the works were carried on in a very intermittent way; but even between these dates a gradual improvement was noticeable in the entrance, as a result of the training-walls constructed. The chart of 1866 showed a 21-foot channel only 150 feet wide, while that of 1907 showed a much straighter entrance; the 21-foot channel had widened

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out to 450 feet, and the sand spit near the end of the north breakwater was "slowly but surely disappearing." As no dredging had been done at the entrance before 1908, these improved conditions must have been the result of the construction of the breakwater. It was often found during the construction of works at river- and harbour-entrances that, owing to gradual alteration of natural conditions, the entrance for navigation purposes was, for a time, even worse than before the works were started. In the case of Newcastle, which was probably the most difficult harbour on the coast of New South Wales to deal with, the improvements, although considerable, had not been so great as might have been anticipated from such extensive works; but Mr. Walsh had no doubt as to the ultimate successful results of the breakwaters as constructed. One of the factors relied upon for the improvement of this entrance was the great scour caused during flood-time, which it was anticipated would scour out the large accumulation of sand that had encroached from the northern side while the works were in progress. In order to show that it was reasonable to rely upon this assistance, it might be mentioned that between the years 1868 and 1898 there were eleven floods in the Hunter River which rose above 30 feet in the gauge at Belmore Bridge, West Maitland; but no such flood had occurred since the north breakwater was sufficiently far advanced to render such scour effective. During the flood of 1893, when the river rose to 37 feet at Maitland, there was a continual flow out of the entrance at 4 to 6 knots per hour for 4 days, the tide-gauge showing a difference of 10 inches to 2 feet between high- and low-water marks. He was confident that, were such a flood to occur now, the whole of the accumulations of sand would be carried out to the coastal current and dispersed, thus nearly doubling the sectional area of the channel between the breakwaters; and as the inflow of sand from the Oyster Bank would be cut off, there seemed no reason to anticipate that the ebb-tide would not be sufficient to maintain the entrance. Considerable difficulty had been experienced during the construction of the first 2,000 feet of the north breakwater, owing to the inrush of beach-sand and the excessive scour at the tip-head; and the sea on the Oyster Bank being very treacherous, it was thought inadvisable to blanket with rubble stone from punts. In 1903 the scour had increased to such an extent as to maintain a channel about 36 feet deep around the tip-head, and the large quantities of sand entering the harbour threatened to interfere seriously with navigation. It was with the object of

Mr. Walsh. arresting this scour at the tip-head and forming a temporary breakwater to keep out the sand encroaching from Stockton Beach that he suggested blanketing the bottom in the line of the breakwater with hulks, as described by Mr. King. The novel suggestion was submitted to an advisory board of engineers, which, having approved the scheme, requested him to superintend the placing of the hulks in position. This he did with the assistance of Mr. King, and, as stated in the Paper, these somewhat hazardous operations were carried out without mishap of any kind. The work proved a complete success, and by reducing the depth at the tip-head enabled the breakwater to be extended at the rate of 80 to 100 feet per month as against 10 feet per month previously. The saving was about £30,000.

With regard to Mr. Halligan's Paper, he thought that all engineers dealing with marine works on the coast of New South Wales would feel indebted to its author for the valuable information he had supplied in connection with the various winds and currents, even though they might not be able to accept the formula he had evolved for their use in designing harbour-works in the future. It would be interesting to know why Mr. Halligan had dealt only with the width of the entrance. Taking, for instance, the case of Jervis Bay, with an entrance 11,590 feet wide and 120 feet deep, the tidal compartment was given as 28,326 acres, and, taking the rise of tide at 5 feet, 6,170 million cubic feet of water passed through the entrance in 6 hours, with a velocity of $\frac{1}{2}$ knot per hour. Did Mr. Halligan contend that this body of water could pass through the entrance if it were only 20 feet deep (instead of 120) without increasing the velocity? If not, surely the sectional area, and not the width alone, must be considered, in order to carry out his theory of keeping the velocity of the flood-tide below that of the coastal current. Mr. Walsh was aware some scientists held that only the upper layers of water need be considered in dealing with a flood-tide entering a river or estuary, but having had considerable experience with divers working at harbour-entrances, he was convinced that the current was at least as strong within a few feet of the bottom as at the surface. At Newcastle entrance, divers were only able to work at slack water on the bottom in 30 to 40 feet of water; immediately the tide began to flow the men were carried off their feet and had to be hauled on board the attendant vessels by their life-lines. It was scarcely necessary to add that only the bottom layers of current carried sea-sand to any extent. If the sectional area of an entrance, and not its width alone, were considered, it was interesting to see how Mr. Halligan's theory with regard to the velocity of the flood-tide worked out. At Newcastle the sectional area between the break-

waters 300 feet north-east of Stony Point was 22,800 square feet, Mr. Walsh. and the flood-tide velocity, as shown by Mr. King (Fig. 10, Plate 6), was 2·6 knots per hour. If, as anticipated, the channel scoured out to its full depth, the sectional area would be increased to about 52,000 square feet, in which case the flood-tide, which now entered at 2·6 knots, would enter at $1\frac{1}{2}$ knot, or less than the velocity of the coastal current off the port. Further, seeing that the wharves were situated immediately inside the entrance, the existing entrance provided better protection for vessels berthed at them than would be the case if the entrance, which was exposed to heavy easterly seas, were made 2,570 feet wide, which would be necessary if Mr. Halligan's formula were adopted. In considering the relative velocities of the ebb- and flood-tides outside an entrance, it must be remembered that the flood-tide did not travel in a direct line from the ocean-current, but entered in a fan shape from all points round the entrance, and therefore the velocity was not so great a short distance out as when well inside the breakwaters. It would be noticed that in Mr. King's chart of 1909 the flood-velocity between the 7-fathom and 8-fathom lines off the south breakwater was shown as 0·5 knot per hour, although it increased to 2·6 knots farther in. On the other hand, the ebb-tide passing out in a direct line between the breakwaters continued in that line until turned by outside influences. In the case of Newcastle, observations showed that the ebb-tide influence was felt for a distance of 800 feet beyond the end of the breakwaters. It would be interesting to hear Mr. Halligan's experience on this point. In the case, say, of the Hunter River, it was somewhat difficult to realize the truth of the statement that upland waters might be disregarded altogether as having no effect on the harbour-entrance. With a watershed-area of 8,269 square miles, and an annual rainfall of 36 inches, it did not seem probable that the discharge from this river gave no assistance whatever to the ebb-tide. Most engineers would no doubt agree with Mr. Halligan's concluding remarks that if his theories were accepted, the future work of marine engineering on the coast of New South Wales would be very simple. He had given, in his Paper, particulars of the coastal current; it would only be necessary, therefore, to send a surveyor and measure the width of the entrance and the area of the tidal compartment. The engineer could then decide whether to increase the width of the entrance and obtain the required 1 foot to 4 acres, or to reduce the tidal compartment, by reclamation or other means, so as to obtain the same proportion.

Mr. Wheeler.

Mr. W. . . . on the north side, from which direction the flood-tide three Papers curved in form and projected beyond the coast-line result of his exp. This river-entrance was entirely free from a bar Australia, that the mass or deeper than any other part of the channel. entirely of sand, was due to find that Mr. Halligan had elaborated a Although this was contrary to the sand were due entirely to littoral the movement of drift-material on sea . . . wave-motion were of no facts brought forward in Mr. Wheeler's Paper the sand enabled the read at the Institution in 1896.¹ The theory as to this act of that which was there set out had been proved to be correct by further experience. In not a single case had he been able to find that the regular and continuous movement of littoral drift was due to the action of the wind. It was generally admitted that the effect of onshore gales on beaches was to tear out the shingle and lower the surface of the beach, and that in calm weather or offshore winds the material so pulled down gradually drifted back again and the beach was restored. Recently he had an opportunity of watching the action of the drift along the shore on the east coast under very favourable conditions. A quantity of ballast for road-construction had been discharged from barges into carts at about half-tide level, and some of it had fallen on the beach. This material was different in character and colour from that which composed the covering of the beach, so that its movement could be traced. At every tide some portion was carried upward and forward along the beach in the direction the flood-tide was taking; and pieces as large as half-bricks were moved in one tide 25 yards upward and 70 yards forward. In the course of a few days pieces of this ballast were to be found at a considerable distance from the place where they had been deposited. The tides were about half-spring, the wind was light, and what there was of it was offshore. It was therefore evident that the movement of this ballast could not be due to wind-action. Another point to which Mr. Halligan drew attention was that along a coast where a littoral drift of sand occurred, and at the same time the shore on the side from which the drift moved extended out from the land on the other side at the entrance to a tidal river, the river-entrance was not encumbered by a sand-bar. Examples of this were cited in Jervis Bay, Botany Bay, and Sydney Harbour. Attention was drawn to the

¹ "Littoral Drift: in its relation to the Outfalls of Rivers and to the Construction and Maintenance of Harbours on Sandy Coasts." Minutes of Proceedings Inst. C.E., vol. cxxv, p. 2.

current, which undoubtedly existed in most of the bays on the coast of New South Wales, was no doubt assisted by the action behind the projecting point to the north of the bays. This eddying action behind the northern head appears to have been overlooked by Mr. Halligan, and carried out for the *Figs. 2b, 2c, and 2d*, did not seem to. Thus, at Newcastle the what took place. The current went far in advance of the north, an eddy would be formed, the sand drifting along the coast would in itself scum, it formed a trap to catch the drifting might result, as stated in Mr. Halligan's Paper and confirmed in Mr. King's, was that enormous quantities of sand were brought into the river-entrance, which had to be dredged out again. Attention was also drawn by Mr. Halligan to the mistake that had been made in contracting the width of the channel unduly where two parallel walls had been constructed, and so throttling the entrance and the scouring action of the tidal water. As an instance of the baneful effect of thus throttling the entrance of the Manning River was cited, where a large bank of sand of marine origin had been formed inside the two training-walls. Taking generally the works that had been carried out for the improvement of the river-entrances described in Mr. Halligan's Paper and shown in the illustrations, in the case of the Richmond River the net result of £420,000 expended had only been to fix the position of the bar in one place; and, as the Author observed, something more might have been expected from the expenditure of so large a sum. In spite of constant dredging, no improvement in the depth of water had been obtained. In view of the inefficient results achieved here and elsewhere, in comparison with the money spent, the question arose whether as good or better results would not have been obtained at less cost by constructing a single wall, as suggested in Mr. Wheeler's Paper on Bars, already referred to, this wall to be placed on the side of the channel from which the littoral drift came, and be carried out into deep water beyond the influence of the littoral drift. The concave side being next the river-channel, the single wall would act as a groyne, stopping the littoral drift from entering the river-channel; the ebb-current from the river and estuary would follow the concave side, while the tidal water entering the river-channel would not be throttled. A good natural example of a connection of a river-channel with the sea carried out on these lines was to be found in the entrance to the River Humber. There a natural breakwater of shingle extended out

¹ "Bars at the Mouths of Tidal Estuaries," Minutes of Proceedings Inst. C.E., vol. c, p. 116.

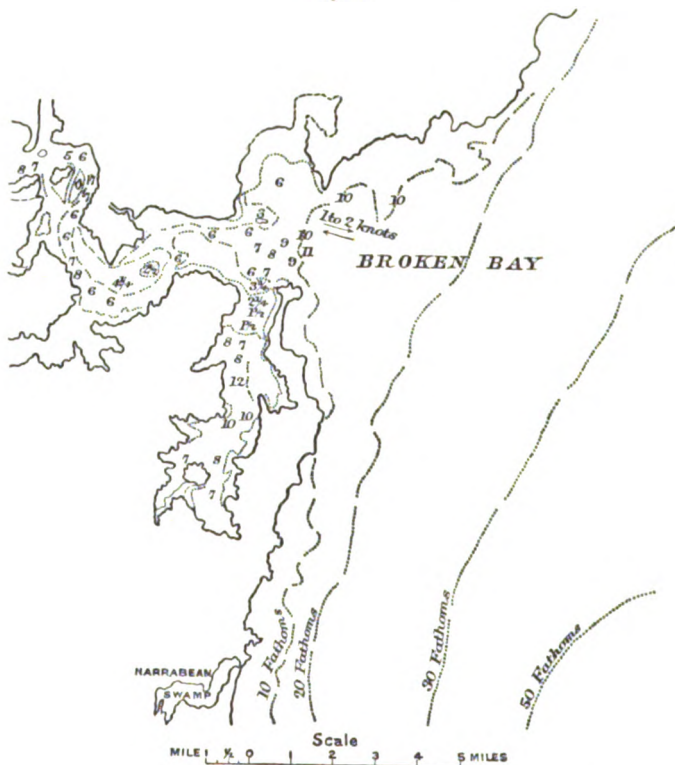
Mr. Wheeler. to deep water on the north side, from which direction the flood-tide came; it was curved in form and projected beyond the coast-line on the south side. This river-entrance was entirely free from a bar and had water as deep as or deeper than any other part of the channel.

Mr. Williams. Mr. C. J. R. WILLIAMS found that Mr. Halligan had elaborated a theory that lateral movements of sand were due entirely to littoral currents, and that the movements due to wave-motion were of no account, except in so far as the disturbance of the sand enabled the littoral current to get hold of and act upon it. In support of that proposition he stated, among other things, that only on rare occasions did storm-waves reach the shore otherwise than normally to that shore. Mr. Williams's own observations elsewhere did not bear out this statement, and it was not likely to be correct in the face of the figures given in the Paper, showing that the movements of north-east and south-east winds exceeding 20 miles per hour in velocity were as 47·4 to 57·9. At Nelson, near the head of Tasman Bay, in the South Island of New Zealand, where Mr. Williams advised on, and for a time superintended the cutting of, a new entrance to the harbour, the harbour was protected by a boulder bank composed of hard stones 6 inches to 12 inches in diameter, which extended for $5\frac{1}{2}$ miles from a rocky bluff, the source of supply. The broken stones found on the beach were quite angular at the bluff, but showed a gradual rounding effect as they receded from the source of supply. Again, at Timaru, on the east coast of the South Island, there were large accumulations of shingle against the projecting south-east breakwater, and before the extension of that breakwater the shingle, $\frac{3}{4}$ inch to 2 inches in diameter, used to be carried into and across the entrance of the harbour in 21 feet at low water. He would be a bold man who would dispute that this movement was due to the prevailing south-east seas, although such littoral current as passed Timaru was northward. Similar conditions existed at Napier Harbour in Hawke's Bay on the east coast of the North Island. The direction of the breaking waves, of course, would depend to a certain extent on the direction of the winds causing the waves, but it was modified by the contour of the adjacent land. At Sumner, a watering-place immediately north of Banks Peninsula, New Zealand, the direction of the breaking wave was always the same, as south-eastern or eastern seas swung round on the peninsula, and the beach lay normal to the seas, having adjusted itself to this condition, as a beach tended to do at all places and at all times. The diagrams of the action of a current passing a projecting headland (p. 130) did not seem quite correct. With regard to *Fig. 2a*, the effect would be somewhat as sketched, and the counter-

current, which undoubtedly existed in most of the bays on the east coast of New South Wales, was no doubt assisted by the eddying action behind the projecting point to the north of each of the bays. This eddying action behind the northern headlands appeared to have been overlooked by Mr. Halligan, and the other diagrams, *Figs. 2b, 2c, and 2d*, did not seem to be fair representations of what took place. The current would be diverted outwards, and an eddy would be formed behind the point. This eddying action would in itself account for the counter-current, which, however, might sometimes be assisted by the action illustrated in *Fig. 2a*. *Fig. 2c* seemed to be particularly incorrect, as in such a case the current could not curl round the point and expand itself on the shore as shown: there would be an eddy, with an outward flow immediately behind the headland, and a counter-current for some distance down the coast, independent of any projecting headland to the south. The description of what took place when the littoral current past an estuary exceeded the velocity of the flood-tide current looked plausible at first glance; but even when the speed of the littoral current past the entrance exceeded that of the flood-tide current into the estuary or harbour, the sand must move along the line of the resultant of the two streams, and some of it must enter the harbour whenever there was a current inwards. At Broken Bay, New South Wales (*Fig. 7*, p. 280), the tidal current seemed, on reference to the Admiralty chart, to exceed that of the littoral current, and one might expect, in accordance with Mr. Halligan's theory, to find the entrance blocked with sandbanks; but the actual conditions indicated that the Bay was filling up slowly from within, while the entrance remained deep. On the other hand, there were indications that some sand was actually carried in by the flood-tide current, and was deposited under the shelter of the headland guarding the south arm, and under the first sheltered point in the main arm. It seemed to Mr. Williams more probable that on this part of the coast the littoral current, which was known to have less velocity than that farther north, was carrying very little sand in suspension, and that this was the explanation of the comparatively slight shoaling in the bays here, rather than Mr. Halligan's ingenious theory. A glance at the chart of the coast of New South Wales indicated that the only place where the counter-current could be ascribed solely to the action indicated in *Fig. 2a* was at the Macleay River (*Fig. 8*, p. 281). There the headland protecting Trial Bay from the south-east did actually appear to divert the littoral current, and the pushing of the mouth of the river northward was perhaps traceable directly to that cause. Mr. Halligan referred to the ascertained

Mr. Williams. fact that during the winter months, when north-easterly weather predominated, the entrances at the southern ends of beaches carried less water than at other seasons. This was exactly what was to be expected, on the assumption that the sand-travel was directly due to the southward component of wave-movement, and appeared to Mr. Williams to tell against the contention that such effects were due entirely to littoral drift. He was in accord to some extent

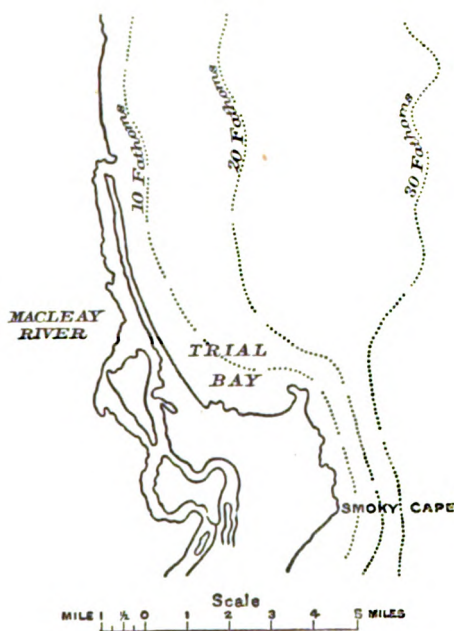
Fig. 7.



with Mr. Halligan in his pessimistic view of the benefits to be obtained from the ebb-tide current; but the engineer could only use what he had, and it must not be overlooked that, where the tidal rise bore a fair ratio to the depth aimed at, the ebb-current at the period of its maximum strength was flowing through a much more constricted area than the flood-current at the time of its greatest velocity. This might sometimes make the ebb more efficient in scouring than the flood, in spite of the fact that the flood-tide had the assistance

of the ocean wave in stirring up the sand; it was especially the case Mr. Williams. when the outlet was partially sheltered from the seas. With regard to the application of Mr. Halligan's principles to the harbour at Newcastle, there were one or two points in which his statements were not in agreement with those of Mr. King, who stated that during floods the Hunter River brought down large quantities of sand, which were swept out to sea. Mr. Williams's experience in connection with the rivers Mary and Brisbane, in Queensland, was that during floods these rivers brought down large quantities of sand,

Fig. 8.



which was deposited in the estuarial compartment. It seemed that there had been some accumulation of sand north of the northern mole at Newcastle, as the shore-line seemed to have advanced about 600 feet since 1866. Finally, one would expect to find, in accordance with Mr. Halligan's theory, a counter-current northward along the Stockton beach; but he showed there a southerly drift of equal velocity to that past the harbour, and seemed to indicate that this was the cause of the sand entering the harbour; while Mr. King attributed it to the flood-tide current. Each case, fortunately for

Mr. Williams. engineers, formed a special study ; and while a knowledge of general principles was essential, and sound theorizing on ascertained data was a help to the proper solution of each problem, no precise rules could be laid down. He did not consider, however, that Mr. Halligan had established his main proposition.

Mr. Palmer. Mr. PALMER, in reply, regretted that Mr. Voisin should have thought that settlement of the south wharf was due in any way to the stone facing. The wharf was built on piles, and it was constructed very rapidly, to meet a pressing demand ; the piles were sunk by means of the water-jet, and it was owing to their settling that the wharf also settled. Mr. Voisin might perhaps revise his opinion as to the cost of the dredging, when it was pointed out that the wages for ordinary labour were 1s. per hour in Western Australia, and all else was more or less correspondingly expensive. Mr. Palmer was indebted to Messrs. Reid Bell, Lefroy, and Shields for kindly placing on record the information regarding the scouring out of the harbour and the washing-down, in a gale, of a portion of the north mole. That the moles would be washed down from time to time had certainly been expected by Mr. Palmer, and also, he was aware, by Mr. O'Connor, who provided very amply in his designs for such washing-down by the sea. Whether what had occurred was more than what was anticipated was not evident from the extract submitted by Mr. Lefroy. The Woodman Point light had been made of a high order and distinctive character in order to prevent the possibility of its being mistaken for other prominent lights along the coast in the vicinity of Fremantle. It was very satisfactory to find an engineer, resident in another State and viewing matters from a disinterested standpoint, speak of Fremantle as a port of which the importance in the Commonwealth was second only to that of Sydney, and consequently consider that the sorting-sidings should have been placed elsewhere. While not seeing reason to disagree with Mr. Keele as to the site actually chosen for the dock, and while concurring with Mr. Lefroy as to the evil effects that could ensue through political influences wrongly exercised, especially as such political pressure invariably meant a wrong sacrifice of the interests of the future and of the State to the illusory present benefit of some locality, Mr. Palmer had nevertheless endeavoured to make his Paper a record of work actually done, and to refrain as much as possible from controversial points ; hence omission from the Paper of all mention of the dock. It might be added, however, with reference to Messrs. Reid Bell and Keele's remarks regarding the site of the dock and the extension of the harbour, that it had been publicly stated in Western

Australia that Fremantle required and deserved reimbursement for Mr. Palmer. removal of the State railway workshops, by construction of a dock within her boundaries. Mr. Palmer's own proposition of 7 years back was a floating dock—a temporary measure, for in his opinion the harbour was not then (and still was not) in a sufficiently advanced state for construction of permanent docks, of which there must be more than one. Mr. Shields's communication showed that there would be no difficulty as regarded depth for the floating dock; it would have been economical and available long since; it could be removed for use elsewhere when the time was ripe for permanent works; and the interest on the capital sum saved meanwhile would have been a clear asset; or, if it had been imperative to expend money, this could have been better spent on warrantable extension of the harbour-works. There was no doubt that, with the aid of wise counsel, Fremantle could be made a great and cheap port; but in order to attain this with benefit to the State as well as to the interested localities, the questions of providing a ship-channel to Perth and of making Fremantle a large entrepôt must be kept separate. It was easy to do this, as they were not, and never had been, conflicting questions. When they were kept separate it would, in Mr. Palmer's opinion, be realized clearly that the extension of the Fremantle Harbour should be to the south-west and not inside the river: the works already constructed lent themselves to such an extension, and from conversations with the late Mr. O'Connor, Mr. Palmer judged that such had been his opinion also.

. Messrs. Halligan and King being in Australia, their replies (if any) to the Correspondence cannot be received in time to be included here, but will be printed in a subsequent volume.—
SEC. INST. C.E.

31 January, 1911.

ALEXANDER SIEMENS, President,
in the Chair.

The discussion upon the Papers by Messrs. Halligan, King, and Palmer, upon Australian Harbours, was continued and concluded.

exactly in the plane of focus of the lens and truly vertical; further, in order that it may be possible to place the pictures round the point from which they were taken at a distance equal to the focal length of the lens, the point in which the optic axis meets the plate must be marked; this point may be called the centre of the picture. If the photographs are intended to give information with regard to contours or levels, the horizontal line through the point just described passes through all the positions shown on the picture which are on the same horizontal plane as the station; and so this line represents the horizon. In this, of course, due allowance is made (if the scale demands it) for the height of the camera above the ground at the station. In all surveying photographs, therefore, a horizontal and vertical line intersecting in the centre of the picture are printed on the plate at the time of exposure. Along the horizontal line may be measured the distances to the feet of the perpendiculars through the objects, while the heights of the objects above or below the horizon (or position of the camera) may be determined from measurements on the vertical. The camera for surveying, then, must be so constructed that the photographic plate before exposure can be set accurately vertical and perpendicular to the optic axis, and also exactly in the principal focus of the lens. There must also be some means for marking at the same time the horizontal and vertical lines through the centre. Finally, the lens must give good definition without any distortion.

Orientation and Mapping.—If now a series of plates has been exposed with such a camera it becomes a simple matter to orient the pictures properly at each station. There are several ways for doing this according to the method of survey adopted.

Suppose that the work is being carried out along a triangulated network of points, the photographic survey being made for the purpose of filling in topography.

Let R, M, S, O, etc. (Fig. 3, Plate 8), represent a series of triangulation stations which have been already plotted on the paper. If the camera is set up at R and a series of plates is exposed, one of which contains the station M, then evidently this particular plate can be oriented either by calculating the angle θ between the axis of the lens and the ray RM, or more expeditiously by making use of a "sight-rule," such as that shown in Fig. 4, Plate 8. In this AB equals the focal length of the lens if the negatives are to be used for plotting the results. If, however, a quarter-plate camera is used a great deal of time may be saved, generally without too great a loss of accuracy, by using prints enlarged, say, three times, and then the distance AB must be equal to three times

the focal length. Measuring now the horizontal distance on the point M from the centre of the cross lines, lay it off $A m$ on the ivory edge of the sight-rule, right or left as the case may be. Then setting B over the point R, bring the point m on to the line through R M produced. The point A is now pricked through, and the line C D is drawn. This line represents the trace of the first picture-plane with the horizontal plane, and may be called the "picture-line"; the line A B is the first position of the lens axis, and is called the "line of direction." Now choose a point N common both to this first picture and to the second, draw its ray R N from the first picture, and then proceed exactly in the same way to orient the second picture. This method with the sight-rule is much more rapid than calculating the angles in each case, and as a rule is well within the limits of accuracy of the map which is being drawn. Of course for very accurate work all measurements must be made from the negative.

As soon as the pictures have been oriented it is easy to begin mapping out the points on the plan. These points are all plotted from intersections of rays in pictures taken at two or more stations, and several methods may be adopted to obtain these intersections. In all cases the first thing to be done is to letter and number in the pictures the points which it is determined to plot. A little care in the beginning of this work will save a great deal of labour afterwards. It is well to arrange the sets of pictures showing the panorama at each of two or three stations, and after careful study first to pick out the most important features, identifying them in the various groups of photographs. A system of lettering and numbering should then be adopted, so that as little time as possible will be spent in finding the proper intersections.

The most expeditious method of transferring the horizontal distances from the prints to the picture-lines is first of all to prick through the points desired, and then to cover the prints with the thinnest and most transparent tracing-paper obtainable, on which are ruled hair-lines to correspond with the vertical lines through the centre of each picture. The points that have been marked on the prints are pricked and numbered on the tracing-paper, which is then transferred to the drawing-board for the survey to be plotted. By keeping the vertical hair-line on the tracing-paper coincident with the line of direction of the picture under consideration, the points are easily marked along the picture-line. With care this method will be found to be generally as accurate as is required by the scale of the map being plotted.

In plotting levels from the photographs great care must be taken

to obtain good results, for it is important to remember when mapping on even such a large scale as 1 inch to the mile that the vertical distance between 100 feet contours if drawn on a section or elevation would be less than $\frac{1}{80}$ -inch. Suppose then that the level of one of the photographic stations to be either known, or to be taken as the datum of the survey. The first thing is to draw the contour-line given by the horizon-lines of the series of pictures taken from this station. The level of any prominent point S, Fig. 5, Plate 8, shown in any of these pictures may then be found, simply by measuring very accurately its height above or below the horizon line, and laying off this height h as shown, at a distance along the ray of S equal to the focal length, or enlarged focal length—according as the original plate or the enlarged picture is used. Then the height h' , drawn through the corresponding point s on the plan, will give, on the scale of the map, the height of S above (or below) the photographic station. In this way the height of some point which appears on the horizon line of a picture taken from another station may be determined, and another contour can be drawn. Having drawn the contours for the horizon lines of the pictures, spot-levels may be determined as just described and the series of contours is then completed. The matter is just as simple if it is necessary to assume some point other than a photographic station as the datum, because always the height of this point can be measured from the horizon line in one of the pictures.

COMPLETE SURVEYS.

Should it be required to make a complete exploratory survey, the azimuth of course must be determined. Since the photographs are taken generally so as to tie together the whole survey, it is only necessary to determine the azimuth of one picture, except for purposes of a check. It will be usually fixed by the magnetic needle, either by noting the bearing of one of the first settings of the camera by a compass fixed on the top of the camera, or preferably by photographing the position of the needle, floating in the camera, at the time the plate is exposed, thus obtaining an automatic record of the azimuth of the picture (Fig. 6, Plate 8). This might be done on each picture, but in practice the direction of the needle is only photographed on as few pictures as will be required to give a proper check of the work; otherwise considerable time would be wasted in waiting for the needle to come to rest each time before exposing the plate.

The exploratory survey may now be rapidly made in the following manner: A base line is first determined by hand-telemeter, or if

the atmosphere is clear and a measuring microscope is available when the survey is being plotted, photographs may be taken of a stadia-rod on which have been made graduations suitable to the scale of the map, and thus the lengths of several suitable bases may be determined. The series of photographs is then taken as already described, the stations for the camera being fixed by intersections.

Example of Survey.—In Fig. 8, Plate 8, is shown a complete survey made from the three known points, 1, 2 and 3, in the Vale of Llangollen. This was plotted in 30 hours, and the photographs, ten in number, were taken in a couple of hours, which time included the selection of the stations.

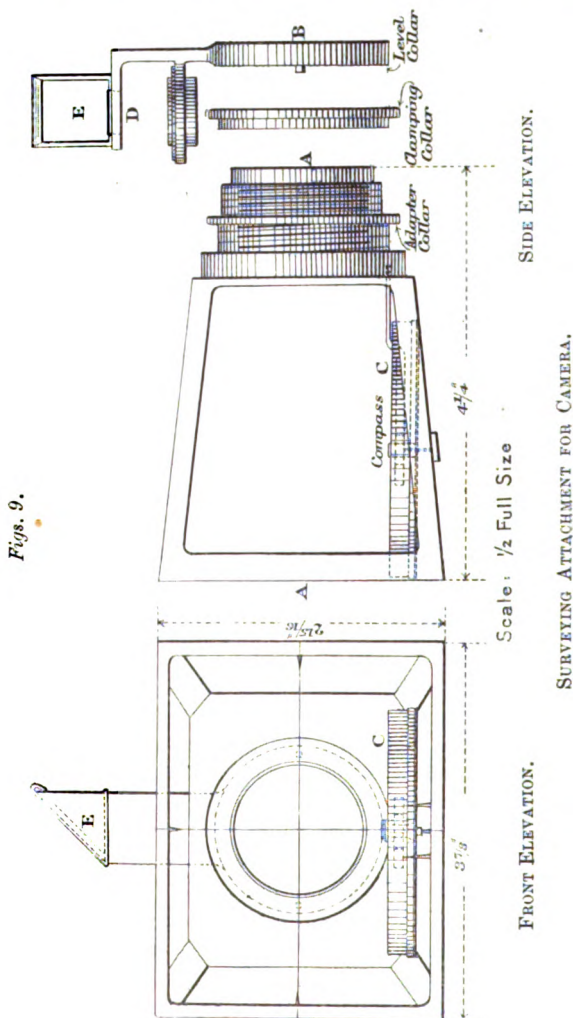
Two of these photographs are reproduced in Figs. 6 and 7, Plate 8, and the letters *a, b, c, d, e* and $\alpha, \beta, \gamma, \delta, \epsilon$ on Fig. 8 refer to the five points, A, B, C, D, E, indicated on Figs. 6 and 7. The scale to which the survey was plotted was 6 inches to the mile, and the photographs were enlarged three times.¹ By comparing the plan with the Ordnance survey one can see how much accuracy it is possible with care to obtain.

APPARATUS.

In photographic surveying the vibrations due to wind offer the greatest impediments to most work. A small camera with short tripod should therefore be used, as offering the least resistance to the wind. Little difficulty is found in practice in levelling the camera when the ordinary tripod is used; being so light, the flexibility of tripod and camera itself is an advantage, and makes the final adjustment of level very easy. As already noted, the essentials are a lens which will give a sharp and undistorted image, and a method of setting the photographic plate accurately vertical in the principal focus of the lens, marking, at the same time that the plate is exposed, the centre of the picture and the horizon. In *Figs. 9* is shown how these requirements have been met in the attachment, for an ordinary quarter-plate camera, designed by Mr. A. C. W. Aldis for the Author. A frame of aluminium, A, carries at one end the lens, and at the other the horizontal and vertical cross-wires at the principal focus. The rectangular end of the frame is made to fit just into the double back of the camera when the dark-slide cover is withdrawn, so that the cross-wires stretched on it can be racked into contact with the

¹ Owing to exigencies of space the map is reproduced at only 3 inches to the mile, and with the photograph enlarged only $1\frac{1}{2}$ times.

plate before exposure. The level is fixed rigidly to a ring B, which has two dowels fitting into the front of the frame A. This frame is first clamped to the front of the camera by the clamping collar



shown, and screwing the lens into the frame then clamps the level also in position, so that with the bubble central the cross-wire frame is vertical. A magnified image of the bubble between two concentric circles ruled on the cover is formed by the lens D

and is seen in the mirror E, thus enabling the final adjustments to be made with accuracy. The floating compass is shown at C. The weight of the whole attachment is only $5\frac{1}{2}$ oz.

A very useful adjunct to the camera is a small number-plate of celluloid, on which, as the plate is exposed, may be written the name of the station and the number of the plate, thus automatically completing the record of the survey.

The Paper is accompanied by seven drawings and ten photographs, from which Plate 8 and the Figures in the text have been prepared.

(*Paper No. 3905.*)

"Tree-Groynes."

By LEONARD PERCIVAL HODGE, Assoc. M. Inst. C.E.

THE system of river-training here described consists in laying in the bed of the river, during the dry season, rows of branches strung on a continuous rope, or detached branches ("Brownlows weeds") anchored separately at intervals, which in the wet season will float and cause an accumulation of silt. The method was first brought to the Author's notice from reading a highly interesting Paper¹ by Mr. C. F. Tufnell, Assoc. M. Inst. C.E., on the methods of training adopted in India.

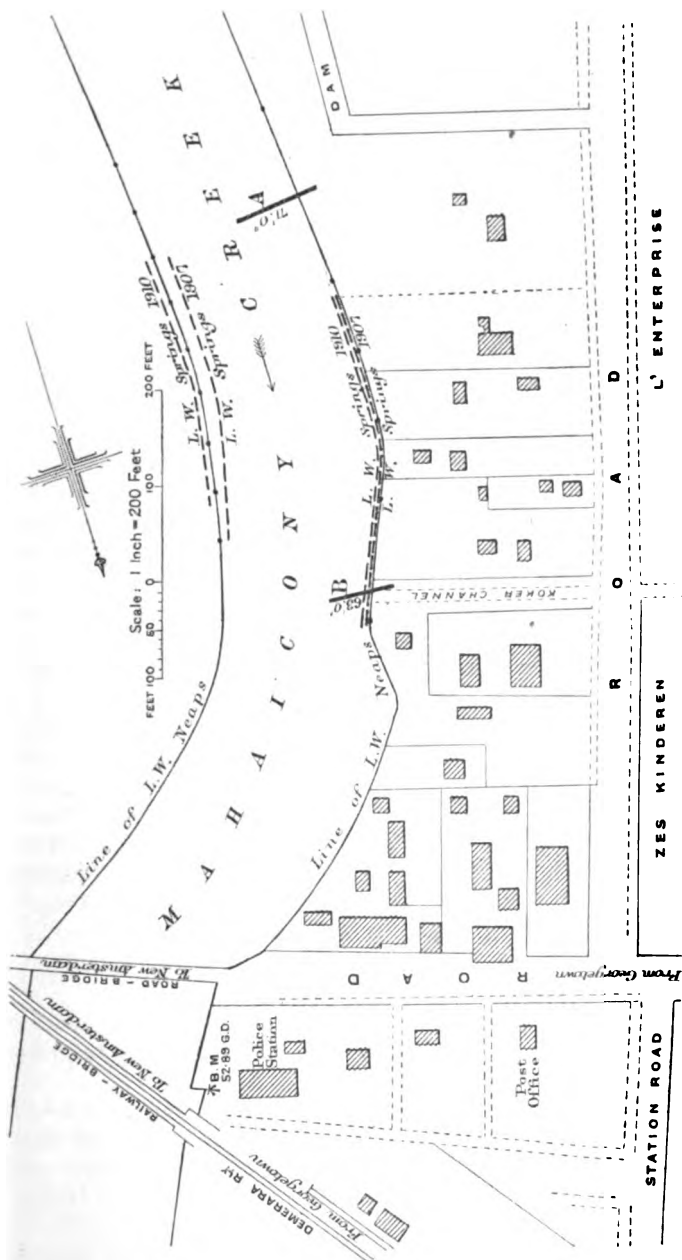
Some modification of the Indian procedure is necessary in countries like British Guiana, where the rivers are not obliging enough to quit their beds during the dry season, and also in cases where the trees most commonly obtainable are so much heavier than the water displaced that they will not float. The silting action is also, at the river-mouth, considerably affected by the tides. The following short account of a successful river-training experiment, conducted at small cost under the preceding disadvantages, will be, it is hoped, of assistance where circumstances are similar.

The Mahaicony Creek, a river about 33 miles long, with a basin of about 400 square miles, had for some years been eating away its west bank, until in 1903 it commenced to erode the village lots; this action, if not checked, would have resulted in cutting across of the principal main road of the Colony and destruction of the embankment of the Demerara Railway, besides considerable damage to both Government and private property (*Fig. 1*). It was decided, therefore, to endeavour to prevent further erosion by throwing out two groynes on the west bank.

The cost was first worked out for groynes of rubble stone, strengthened by greenheart piles and walings, but owing chiefly to

¹ "Economical River-Training in India." Minutes of Proceedings Inst. C.E., vol. lxxii, p. 177.

Fig. 1.



the cost of transport the estimate amounted to \$8,000 (£1,642). A second estimate, substituting sheet-piling with clay obtainable in the vicinity rammed between, amounted to \$4,000 (£821). The area immediately affected was only slightly over 9 acres, with an estimated value, including the erections thereon, of \$20,837 (£4,280), so that even the lower estimate would have imposed a considerable burden upon the assessment of the properties to be protected.

The Author, having noticed during his survey that the Creek water carried considerable sediment in suspension, and that both banks were thickly belted with courida trees (*Avicennia nitida*), suggested that before incurring heavy expenditure on permanent or quasi-permanent groynes, an experimental groyne of courida trees should be constructed on the lines of Indian procedure. He proposed that the trees should be obtained from the east bank opposite the village, so as to aid erosion there, and that, as courida trees will not float, they should be dropped into the Creek at low water in layers, lapping longitudinally to form a groyne. The trees in each layer were to be connected by No. 5 S.W.G. galvanized-iron wire, and each tree was to be weighted by two boulders, weighing about $\frac{1}{2}$ ton each, so as to keep the groyne approximately in position. The number of trees in each layer was to decrease from the bottom upwards, the cross-section of the groyne being thus roughly triangular. His suggestion was approved, and £46 was voted for the experiment.

The experimental groyne was laid down at A (*Fig. 1*), about 50 feet of it being submerged at low water of spring tides. To ascertain its effect on the east bank poles were driven into the creek-bed there at 50-foot intervals (as shown by the dots in the Figure) in the actual line of low-water neaps, and measurements were taken therefrom to dead low-water line at the following springs (19th May, 1905). These measurements were repeated 17 days after, when tide-level was 0.04 foot lower. Despite the reduced water-level it was found that dead low-water line had advanced eastward on the east bank an average distance of $4\frac{1}{2}$ inches.

Silt also commenced to accumulate on the west bank to the immediate north of the groyne, thus demonstrating the success of the initial experiment.

It was thereupon decided to raise groyne A and extend it farther into the Creek, thus inducing increased erosion on the east bank, and a similar groyne was also constructed at B (*Fig. 1*). The work was estimated to cost \$1,100; it actually cost \$1,095.16 (£225), and was completed early in 1906.

By December, 1906, the original neap-line poles on the east

bank had all disappeared. They were round hardwood spars about 3 inches diameter and 3 feet 6 inches long, driven 3 feet into the soft clay of the creek-bed, and were probably washed out as the bank eroded. Fresh poles were inserted on the 5th April, 1907, 10 feet long, and driven about 8 feet on the actual line of low-water neaps. The lines of low-water spring tides for both 1907 and 1910 are shown by the dotted lines in the Figure. The latter, taken on the 11th April, showed that during the 3 years this line of spring tides had advanced an average of 20 feet on the east bank and receded an average of 3 feet on the west bank, the area lost on the east bank, as far as can be gauged from the poles remaining, amounting to 5,083 square feet, and the area gained on the west bank being 1,996 square feet. The erosion of the west bank is still continuing south of groyne A, thus showing incidentally that the groynes are responsible for the making up north of them.

The first effect of groyne A, which would naturally act somewhat as a weir, was to clear away the soft mud. The second soundings of the 5th June, 1905, showed that the west bank had commenced to make up and the east bank to erode. Those of the 30th November, 1906, showed a general accretion and those of the 27th April, 1907, a general loss, the latter due to the constant ebb of the Creek in flood during an exceptionally heavy and prolonged rainy season. But even then the loss was greater on the east than on the west bank, showing that the groyne was performing its work properly. The next soundings of the 16th March, 1908, showed a further erosion on the east and a gain on the west. The last soundings of the 16th April, 1910, revealed a still further erosion on the east bank and a loss on the west, the latter probably due to a recurrence of the weir-action noticed in the first instance.

The actual loss of land on the east bank and its gain on the west bank are not in themselves material; the principal factor in cases such as this being the relation of estimated cost to capital value. To have protected the frontage included in the scheme, about 800 feet, by continuous stone pitching was out of the question as the cost would have equalled, if it did not exceed, the value of the property protected.

The diversion of the Creek by permanent stone groynes would have meant a tax of 38 per cent. on the value of the property, and timber and earth groynes, while considerably less permanent, would have entailed a charge of 19 per cent., exclusive of maintenance. Heavy expenditure on groynes of a permanent, or quasi-permanent, nature is also open to the objection that their effect can never be absolutely foreseen, and that at best they are

useless as soon as they have achieved their object. If groynes therefore are decided upon in preference to direct protection of the foreshore, the chances are that permanence will be no advantage if purchased at a considerable increase on the cost of temporary groynes, provided always that the local conditions favour the employment of the latter.

In this case an expenditure of \$1,320 (£271), representing 6½ per cent. of the capital value protected, has answered sufficiently well. Stone, or timber and earth, groynes might have done the work better, or more quickly, but at an expenditure which was practically prohibitive.

The Paper is accompanied by three tracings, from which the Figure in the text has been reproduced.

(Paper No. 3945.)

"Experiments on the Flow of Water over Sharp-Edged Circular Weirs."

By HAROLD JOHN FREDERICK GOURLEY, B.Eng., Stud. Inst. C.E.

As a means of measuring flowing water, the sharp-edged circular weir is quite a recent innovation, the only application known to the Author being that in the works, the first instalment of which is now nearing completion, in the Derwent Valley. In the completed scheme, there will be a total length of 400 feet of circular weir in diameters of 3 and 6 feet. When the advantages possessed by the circular weir are better known, there can be no doubt that its use will be greatly extended. Such a weir is much more compact than is the corresponding straight weir, and on this account may be used in many places where the equivalent straight weir would be out of the question. The experiments show that the law of flow holds for a wide range, and thus a comparatively small weir may serve to discharge quite a town's supply. As an example, a weir 4 feet in diameter would discharge about 14 million gallons per day under a head of 9 inches from a chamber 12 feet square.

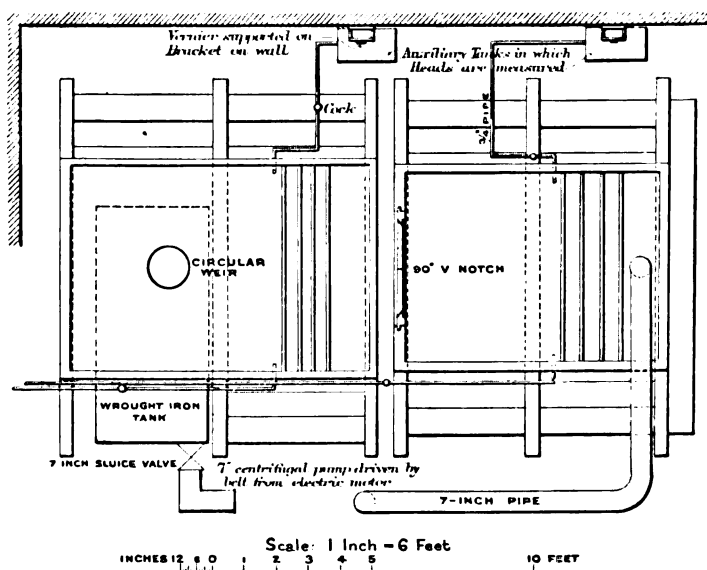
Circular weirs to discharge up to 15 million gallons per day are readily and accurately made from ordinary cast-iron flange and spigot pipes by turning an edge on the spigot itself or else rebating the spigot to receive a gun-metal lip which is secured by counter-sunk screws. Although weirs to discharge larger quantities are unlikely to be required, they may be obtained by using steel tubes.

General Arrangements.—It was intended to carry out the experiments on as large a scale as possible, and with this end in view the Author adopted the following general arrangements.

The absolute measurement of the quantity flowing was obtained by passing the water through a sharp-edged 90° V notch, shaped in the end of a reinforced-concrete tank, the bottom of the V notch being 2 feet 7 inches above the floor of the tank. The V notch was used because it is undoubtedly the most accurate means of measuring flowing water. The Author was fortunate in having

at his disposal the results obtained by Mr. Barr in a very accurate and extensive investigation of the flow through such a notch carried out in 1908-1910.¹ The water fell into a second reinforced-concrete tank and flowed over the circular weir, the lip of which was about 2 feet above the floor of the tank. After falling into the lowest tank, placed on the floor, the water was pumped again to the upper tank by a 7-inch centrifugal pump driven by belting from a shunt-wound electric motor. There was practically no vibration from the machinery, but as a precaution the delivery pipe was suspended quite independently of the upper tank. Ample regu-

Fig. 1.



OUTLINE PLAN SHOWING GENERAL ARRANGEMENTS.

lation of the flow was obtained by varying the opening of a sluice-valve placed between the lowest tank and the pump.

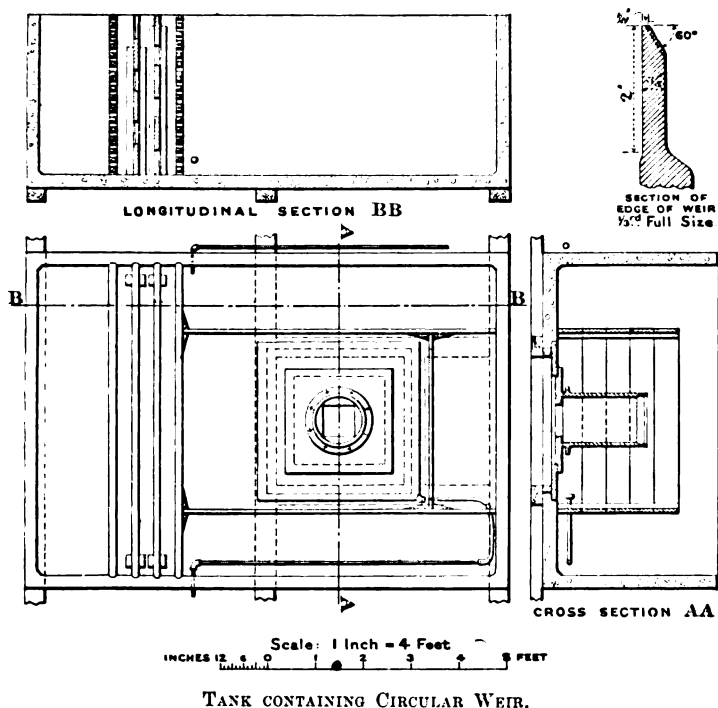
The upper tank was connected by $\frac{3}{4}$ -inch piping with a small auxiliary tank, 1 foot 10 inches long by 1 foot 2 inches by 10 inches deep, mounted upon an independent trestle, as shown on the general plan (Fig. 1), and in this way it was possible to measure the head over the V notch in perfectly still water. A very fine-pointed brass-rod was used to take the water-level in the small tank. To it

¹ *Engineering*, vol. lxxxix, 1910, pp. 435 and 473.

was attached a vernier moving along a steel scale so that direct readings to $\frac{1}{100}$ inch and estimated readings of $\frac{1}{1000}$ inch were obtained. The vernier and scale were attached to brackets fixed to the wall. An exactly similar arrangement was used for measuring the head of water over the weir. The whole of the arrangements were such that one person could take all the necessary observations with ease.

Construction of Reinforced-Concrete Tanks.—Considerations of cost showed that reinforced concrete was the cheapest material to use,

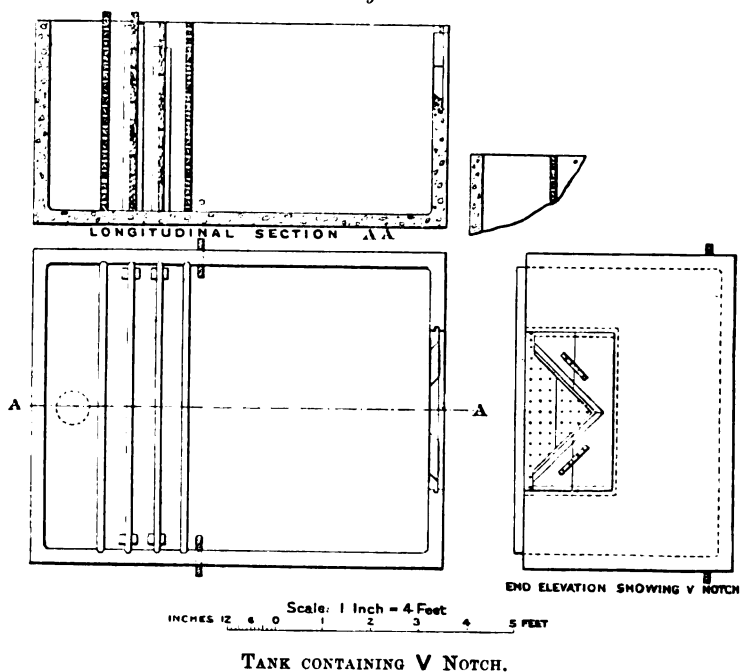
Figs. 2.



and as these tanks took the Author 3 or 4 months to make and equip, their construction will be briefly described. The lower tank was constructed first, and after shortening, the shuttering boards were used for the upper tank and finally formed the platforms round the tanks and the baffle-plates. The arrangement finally adopted for the baffle-plates is shown in the sections (*Figs. 2 and 3*) and was quite satisfactory; at maximum flow the fall in

level through the plates was not more than 2 inches in either tank, and yet the eddies at entrance were most effectually damped. The upper tank was 8 feet long by 6 feet by 4 feet deep inside, the sides and floor being $3\frac{1}{2}$ inches thick. The concrete used was 4 : 1 ; the aggregate being Fleetwood gravel from which all stones having a dimension greater than $1\frac{1}{2}$ inch were excluded. It was mixed rather wet and for the walls was put in continuous shallow layers and moderately punned. The reinforcement in the sides consisted

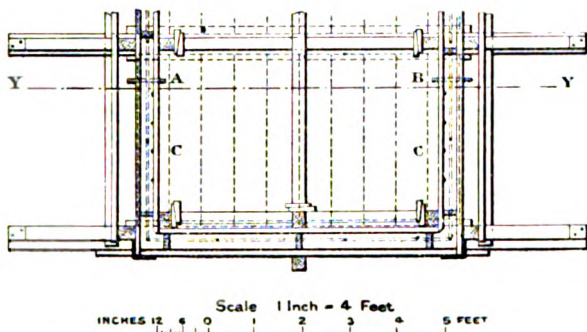
Figs. 3.



of bands of $\frac{7}{8}$ -inch by $\frac{1}{8}$ -inch hoop-iron at vertical intervals ranging from 5 inches at the bottom to 10 inches at the top, with vertical iron rods $\frac{5}{16}$ inch in diameter, about 2 feet apart, passing through $\frac{1}{2}$ -inch holes in the hoop-iron bands. The grooves for the baffle-plates were formed during construction by oiled hardwood strips each attached to the bottom inside board by a screw and held vertically and hard up against the remaining boards by temporarily screwing the upper end to a rail of wood placed above the finished top-level of the wall. By building in the two pieces of $\frac{3}{4}$ -inch

pipng shown, provision was made on the one side for emptying the tank, and on the other for measuring the head of water. This tank was constructed so that the bottom was 5 feet off the floor,

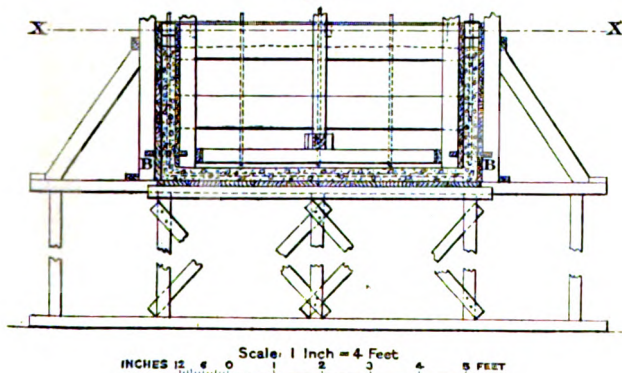
Fig. 4.



HALF PLAN SHOWING SHUTTERING BELOW XX (Fig. 5).

the trestle heads projecting beyond the sides of the tank. This projection was useful during construction and served later to carry the platform which ran along the sides of the tank. The details of the trestles and method of shuttering, concreting and reinforcing,

Fig. 5.



SECTION ON YY (Fig. 4).

will be clear from *Figs. 4 and 5*. Ordinary galvanized wire meshing, 1-inch holes, was laid on the lower part of the concrete forming the floor, and on top of this iron rods $\frac{5}{16}$ inch in diameter were laid longitudinally and transversely about 18 inches apart, the ends of

the rods being turned up for $\frac{3}{4}$ inch. A covering layer of concrete about 2 inches thick completed the floor. The boards forming the shuttering were well soaped before using, and beyond a little lipping due to twisted boards of slightly varying thickness the concrete faces were smooth. The shuttering was eased 2 days after the completion of the concreting and struck a week later; during this time the concrete was watered, and, in fact, until the tank was brought into regular use, the hose was turned on it every other day. The walls and floor were given two washes of neat cement grout, and the tank proved to be quite watertight and satisfactory in every way.

A rectangular gap was left in the end, having a $\frac{3}{4}$ -inch by $\frac{3}{4}$ -inch groove running round $1\frac{1}{4}$ inch from the inner face and the tongue attached to the 2-inch wooden frame in which the V was cut fitted into it, a watertight joint being made with red and white lead. A 2-inch by $\frac{1}{8}$ -inch brass strip with a sharp edge was let flush into the face of the wood, and with its sharp edge projecting $\frac{1}{2}$ inch beyond the bevelled edges of the wooden V. Great care was taken to see that the angle of the notch was 90° , and that when fixed the two sides of it were equally inclined.

The other tank was 9 feet 6 inches by 6 feet 6 inches by 3 feet 4 inches inside, with sides 3 inches and floor $3\frac{1}{2}$ inches thick constructed in the same manner as the upper tank. This tank was also on trestling and stood 3 feet off the floor.

In the experiments, circular weirs ranging from 7 inches to 27 inches in diameter were used, and to accommodate these different sizes a wooden shutter of the form shown in *Fig. 3* was made. The sides of the square hole in the bottom of the tank were supported by 5-inch by 3-inch timber trimming. The 2-inch by 2-inch margin shown was screwed to the trimming, and the shutter when bedded on a packing ring of spun yarn laid on the concrete was screwed down to the margin. The cast-iron pipe which formed the weir was bolted down to the shutter, a packing ring of oiled sash cord having been previously placed between the shutter and the face of the flange. By using a thick packing such as sash cord, it was possible to level up the weir very accurately by tightening the bolts. The joints between the concrete and the shutter and between shutter and pipe were made watertight by clay puddle. For the smaller pipes it was only necessary to screw down other shutters on the one already fixed.

Weirs.—The weirs investigated were obtained by turning up the spigot ends of 6, 9, 12, 18 and 24-inch flanged cast-iron pipes of an average length of 2 feet. The sections of the lip were initially the

same in every case and are shown in *Fig. 3*. The outer periphery was trued for not less than 2 inches from the end. The end was next trued, and leaving $\frac{1}{8}$ -inch square a 60° bevel was turned on the inside and finally the thickness of the spigot was reduced to $\frac{3}{8}$ inch for a depth of 2 inches measured from the lip. For further experiments, the $\frac{1}{8}$ -inch square on the 6-inch and 9-inch pipes was reduced to $\frac{1}{2}$ inch, the 60° bevel being maintained. As it was found that the turned-up surfaces of the lips became coated with rust after but little use, they were wiped with a slightly oiled cloth before commencing experiments.

A TYPICAL SERIES OF EXPERIMENTS.

Before beginning any series of experiments on a particular weir, the zero-levels of the **V** notch and of the weir were taken. The cock of the pipe connecting the upper tank with its auxiliary tank being open, the water in the main tank was raised until it stood about $\frac{1}{8}$ inch above the bottom of the **V**. The water-surface at the notch had then a pronounced curvature, and when an electric light was held some distance above and behind the notch, a number of rings concentric about the bottom of the notch were visible on the surface. The cock on the outlet pipe was then opened, so that the water-surface fell very slowly; as it did so the circles spread out and finally disappeared; at this point the cock was shut. The water-surface was now a flat plane and level with the bottom of the notch. Readings corresponding with the level of the water in the auxiliary tank taken after 3 minutes and again 3 minutes later being averaged gave the zero reading. This observation was repeated two or three times according as the first two differed little or much and the mean was recorded as the initial zero reading. Such observations never differed by more than 0.005 inch, and on the average agreed to about 0.003 inch. A vertical wooden scale was attached to the baffle-plate near the side of the tank, so that the level of the water in the tank as read on the scale corresponded with the reading recorded for the small tank.

When making the joint between the circular weir and the wooden shutter, the weir was set approximately level by using a spirit-level and steel straight-edge. The final levelling was done when the water in the tank itself could be taken as the datum, and it will be seen that great accuracy could be obtained in this way.

When the water had risen to within $\frac{1}{4}$ inch of the lip, the latter was wiped quite dry and the water was raised until it stood about

$\frac{1}{8}$ inch above the edge all the way round. The outlet cock was then opened so that the water very slowly fell. An electric light was moved radially across the lip and a few inches above it at various points round the circumference. In this way after a little practice it was found possible to detect when the water-surface was exactly level with the plane of the lip and the outlet cock was then closed; the addition or subtraction of half a cupful of water at this stage made quite a visible change in the water-surface at the lip of the weir. The mean initial reading was obtained from observations which never differed by more than 0.002 inch on the average and 0.003 inch as the maximum and was recorded as already described for the V notch. It should be mentioned that if at the time the zero was to be taken it was found that the weir was not exactly level, it was made so and then the zero was obtained. In general, the zero readings of the notch and weir were only taken immediately after fixing the weir and just before taking it out. A wooden scale was also fixed in this tank.

Before actual experiments began, several trials were made, and as a result one or two modifications were introduced. It was found that at low heads marked pulsations occurred in the water flowing over the V notch. These were found to be due to the small depth of submergence of the end of the 7-inch pipe through which the water entered the tank, and by tying a sack over the mouth of the pipe exceedingly steady conditions were obtained. At heads above 4 inches over the V notch, no sack was required. It was also found that the first arrangement of baffle-plates did not damp the eddies satisfactorily at the greater flows, and it was therefore altered. In the first instance the water was sent through two sets of plain baffles and then through two sets of drilled boards; in the final arrangement the order was, one set of drilled boards, two sets of plain boards, and lastly the other set of drilled boards (*Figs. 2 and 3*). The trials frequently extended over several hours, and it was found that at the various openings of the sluice-valve the flow remained constant over periods of $\frac{1}{2}$ hour and more.

By introducing a suitable air-pipe, the pressure beneath the lip of the circular weir was always atmospheric, and the Author would here emphasize the necessity of always making some such provision in practice. In this case, the pipe was only required at low heads, but in most cases in practice it will be needed for all heads.

The procedure in an actual experiment is as follows. The cocks on the pipes connecting the main tanks with the auxiliary tanks are opened, the pump is primed and started with the sluice-valve opened

as much as is necessary to allow the minimum quantity of water to flow in the circuit, this minimum quantity having been previously and experimentally determined as the smallest quantity which the weir will discharge without the water leaving the jet formation and beginning to cling to the interior of the pipe from top to bottom. The scales having been read in the two main tanks, as much water is added to the auxiliary tanks as will bring the water in them as indicated by the verniers to practically the same level as that in the corresponding main tanks. An interval of 10 minutes is then allowed to elapse, during which any small difference between corresponding tanks adjusts itself. Exact readings of the verniers are next taken, each reading taking 10 to 15 seconds, and after an interval of 4 minutes a second set is taken; if the two sets do not agree closely another is taken after an interval of 4 minutes. The actual observations are recorded in the form indicated in Table I of the Appendix, and from these sets of readings the results recorded in Table II are obtained. Tables I and II are the results of an actual experiment, and may be taken as typical of the others. The sluice-valve is then opened a little more and the same operations are repeated.

The maximum flow to be recorded for the smaller weirs was determined by the point beyond which the law of flow was found to alter, i.e., the head at which there was marked interference and possibly eddying at the spot where the upper surface of the inflowing water intersected the axis of the pipe. In the case of the two larger weirs the upper limit was imposed by the power of the electric motor, i.e., when the flow reached about 3 cubic feet per second.

In some of the earlier experiments the temperature of the water in all four tanks, and of the weir also, was taken before and after each series of experiments. The temperature differences were never found to exceed 3° F. between corresponding tanks, and, as the head connection for this worked out to 0.00025 inch—a quite negligible quantity—no account has been taken of temperature differences. The temperature of the water averaged 67° F. throughout the whole series of experiments, ranging from 64° to 70° F.

Table II of the Appendix contains a summary of the readings corresponding to Table I, together with the logarithms of the two heads. From these a curve connecting the logarithms of the heads over the notch and weir was plotted on a large scale as the experiment proceeded, and any experiment whose results lay off the general straight line given by the points was repeated.

The results given by Mr. Barr show that the value of the index in
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the formula giving the flow over the **V** notch is constant and equal to 2.5. The law of the circular weir was also expected to be of the same form as that of the **V** notch, i.e.—

$$Q = K L H_w^m.$$

$$\text{Therefore } Q = C H_v^{2.5} \text{ for the } \mathbf{V} \text{ notch}^1 \quad . \quad . \quad . \quad (1)$$

$$\text{and } Q = K L H_w \text{ for the circular weir} \quad . \quad (2)$$

Equating (1) and (2) and taking logarithms of both sides:—

$$\log C + 2.5 \log H_v = \log K + \log L + m. \log H_w$$

$$\therefore \log H_v = \frac{2}{5} (\log K + \log L - \log C) + m \frac{2}{5} \log H_w \quad . \quad . \quad (3)$$

In equation (3) all the units are feet. If h_v and h_w , the heads measured in inches for the **V** notch and circular weir respectively, be substituted for H_v and H_w the equation would read:—

$$\log h_v = (\log K + \log L - \log C + 2.5 \times \log 12) + \frac{2}{5} m (\log h_w - \log 12) \quad . \quad . \quad . \quad . \quad (4)$$

Equation (4) shows that if the values of $\log h_w$ be plotted as abscissas and of $\log h_v$ as ordinates a curve is obtained, the slope of which is $\frac{2}{5}m$ and from which therefore the value of m can be readily deduced. The value of L , C , and m being known, the value of K can be at once obtained.

Accuracy of Results.—It was possible to measure the level of the water in each small tank to 0.002 inch. Observations of the zero-level of the **V** notch differed at most by 0.005 inch. As the results were averaged the maximum possible error would be 0.0035 inch. This would make a percentage error in the smallest heads, viz., about 2.5 inches, of ± 0.14 per cent. and the corresponding error in the quantity discharged would be ± 0.35 per cent. In the case of the smallest circular weir, the maximum possible error would be 0.0025 inch and this would make a percentage error in the minimum recorded head 0.40 inch, of ± 0.63 per cent., or in the quantity, ± 0.88 per cent. For the largest weir the lowest head recorded was 0.60 inch and the corresponding errors would be ± 0.43 per cent. and ± 0.60 per cent.

¹ The value of C for a series of experiments was taken as constant and equal to the mean for the range of flow in the series.

The diameters of the weirs were measured with great care in three directions making 120° with one another. The maximum difference found in any case was less than 0.1 per cent. and the averaging reduced any error to quite a negligible quantity. The curves would also reduce the probable error in the results given in the Appendix. Individual results are believed to be accurate to ± 1 per cent., while as the result of the curves the limit of error for any series of experiments is probably reduced to ± 2 to 3 per cent.

Résumé of Experiments.—The actual diameters of the weirs investigated were 6.91, 10.08, 13.70, 19.40, and 25.90 inches, and the experiments were made under the following conditions. After a series of experiments had been made on the two smaller weirs, their edges were reduced from $\frac{1}{8}$ inch to $\frac{1}{32}$ inch and another series was carried out.

The head over the circular weir was in certain cases measured just where the $\frac{3}{4}$ -inch pipe connecting the large tank with the small one left the tank-side near the baffle-plates and in a corresponding position when false sides were in the tank. The results obtained in this way are entered in the Tables under the heading "Head measured at side." In other cases the measurement was taken in the angle of the chamber where the side joins the end opposite the inlet, and the results so obtained are found under "Head measured opposite." The connection between the desired point in the reduced chamber and the pipe through the tank-side was made by further iron piping and flexible hose as indicated in *Fig. 3*.

In practice the question of the minimum size of chamber to contain a given weir is important, and an arrangement of adjustable false sides was introduced as shown in *Fig. 3*, by which it was possible to reduce the chamber from 6 feet 6 inches to about 3 feet square. The effect of these reductions was observed both when the false sides and end were in, and when the sides alone were fixed.

Further experiments were made with a view to find how the law of flow was affected by variations in the width of the channel of approach to the weir-chamber. This reduction was effected by covering the holes in the baffle-plates by vertical sheets of zinc, thus leaving a passage which could be reduced in width by increasing the breadth of the sheets.

Mr. Barr measured the heads in the corner formed by the end containing the notch and the side of the tank. The heads in these experiments were measured at the side, with the exception of experiment 47b, which was experiment 47a repeated with the head over the V notch measured in the angle. The results of these

experiments show that there is no practical difference caused by measuring the head at the side instead of at the angle.

Explanation of Tables III-VII.—These are the records of experiments made upon the 6, 9, 12, 18, and 24-inch diameter weirs respectively. The experiments made with the reduced chambers are found under the heading "False sides and end," or "False sides" and the figure given in the distance column is the distance between the outer circumference of the weir and the shutters. The term "reduced passage" will be clear from what has gone before; the figure in the distance column is the width of the passage. When there are neither zinc sheets nor wooden shutters in the tank the results are found under the heading "All clear."

DISCUSSION OF RESULTS OBTAINED.

Effect of Reducing the Thickness of the Lip.—The results of experiments 1 and 2, 13 and 14 show that the reduction does not alter the law. It would appear that once the water has passed the outer edge of the weir the thickness of the lip used in practice is immaterial; but one would expect to be able to pass smaller quantities of water over a fine edge than over a blunt one without clinging taking place, and this is what the Author observed. For example, the minimum head observed on the 9-inch weir with the thick edge was 0.65 inch, whilst it was 0.38 inch with the thin edge: the corresponding heads for the 10-inch weir were 0.55 and 0.40 inch.

Differences Due to Position in which Head over Weir is Measured.—Reference to Tables III-VII of the Appendix shows that there are variations, mostly slight, but in one or two cases more noteworthy. Experiments made with "velocity rods" indicated that in every case the water in the angles of the chamber remote from the baffle-plates was "dead" or "still," and bearing this fact in mind, the Author thinks it better to consider the results obtained by measuring the head in one of these angles rather than the others when deducing the average laws for each weir. By so doing, some similarity of conditions is obtained. Nevertheless, in certain cases the side results will be useful for comparison.

Effect of Reducing the Size of the Chamber.—The reduction was effected by introducing close-boarded wooden sides and ends. Considering experiments 24, 27, 28, 29 and 30 of Table IV, it will be noticed that the coefficients decrease from 3.06 to 2.90 and then increase to 2.98 whilst the index is 1.42 for experiments 28, 29

and 30, and increases to 1.43 and 1.44 for experiments 24 and 27. The effect of the increased coefficients is reduced by the increased indices so that the maximum difference is only 2 per cent. between experiments 24 and 28 or 29. It therefore appears that the effect of the size of the chamber on the law of flow is small, the minimum flow for a given observed head occurring at a certain distance, in this case 13 inches. It appears that there is some small physical effect due to the sides beyond this distance and that within it the law so alters that for a given measured head a greater discharge is obtained. Doubtless this comes about thus: As the water is very confined, the surface curve instead of becoming flatter and more nearly horizontal as the distance from the weir increases, becomes bulged, so that the head as measured in the angle is less than the maximum head causing the flow over the weir and as the measured head is less than it should be, the modification referred to takes place.

The results of experiments 17 to 23 (Table IV) indicate that in these cases the ends have no appreciable effect on the flow, and from this it would appear that, within the limits of the experiments, it is mainly the restriction caused by the sides which brings about the changes in the law. One interesting feature in the preceding experiments may be remarked upon here. It might be expected that any effects due to a reduced chamber would begin to appear beyond a certain head depending upon the size of the chamber. The curves obtained, however, show no trace of this and the law in every case holds over the same range. Similar results to those just discussed are given in Table V. In this case the limiting distance is about 18 inches.

Effect of Varying Width of Approach Channel.—Alteration of the width of the channel of approach causes evident variations, and it is clear that if the passage be unduly restricted, the departure from the general average law is marked. This appears to be caused by the velocity of approach being so great as to shoot the water over the weir without first giving it time properly to surround the weir.

Deduction of Laws.—For reasons already stated, it is considered better when deducing the laws of flow for each of the weirs to take the results obtained by "measuring the head opposite." In the case of the 6.91-inch weir, one result only has been obtained by measuring the head in this way—No. 3—and the index has the value of 1.39. As will be seen later, this is low and the Author is unable to account for this fact. If the index be increased to 1.42

the values of K giving agreement at lower and upper limits of observations are 2.88 and 2.98. Therefore, taking $K = 2.93$ gives a maximum error of 1.7 per cent. In this way the formula for the 6.91-inch weir will become $Q = 2.93 L H^{1.42}$.

Turning now to the results obtained for the 10.08-inch weir as given in Table IV, there is no doubt that the index has the value 1.42. As before, the values of K to correspond with index 1.42 in experiments 24 and 27 are 3.00 and 2.91 respectively, giving an error of 1.7 per cent. Omitting No. 26, in which there is a marked variation caused by the narrow inlet, the mean of the other six values of $K = 2.94$. The average formula for the 10.08-inch weir is thus $Q = 2.94 L H^{1.42}$.

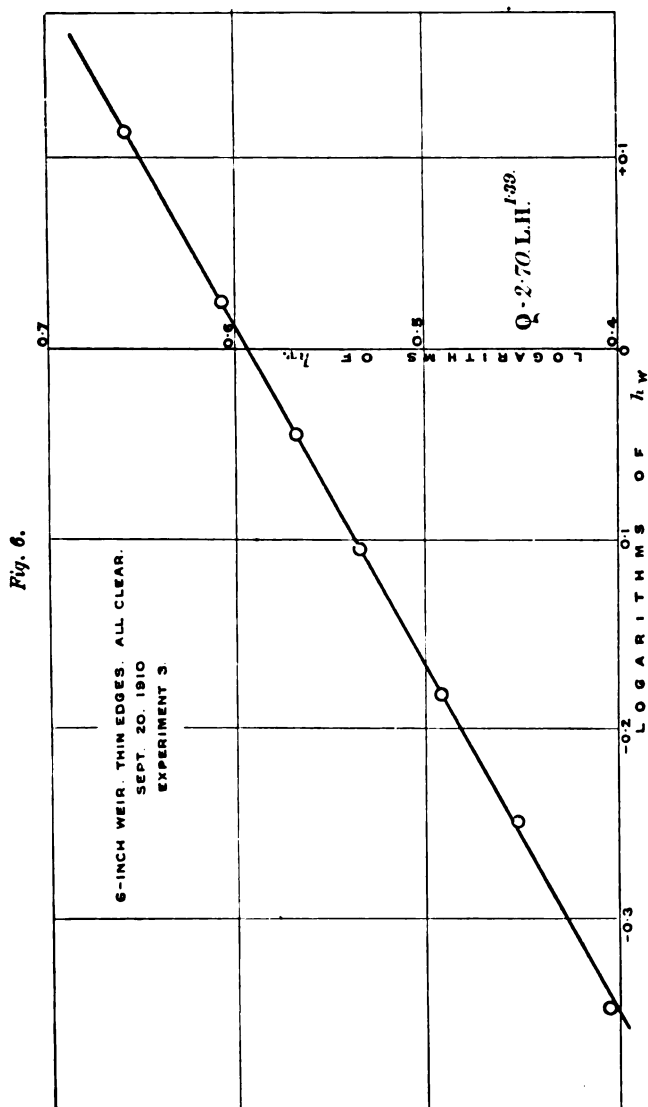
From Table V the value of the index m is clearly 1.42. By increasing the index in experiment 39 to 1.42, K becomes 2.97, with a maximum error of 1 per cent. Omitting the result of experiment 43, in which the proximity of the sides has had a noticeable effect, and taking the mean of the others, the average law of the 13.70-inch weir is $Q = 2.97 L H^{1.42}$. Proceeding on similar lines, the average law is $Q = 2.99 L H^{1.42}$ for the 19.40-inch weir, and $Q = 3.03 L H^{1.42}$ for the 25.90-inch weir.

The foregoing values of the constants, viz., 2.93, 2.94, 2.97, 2.99 and 3.03 show that K increases very slowly with the diameter. A simple rule appears to give the values of K for these experiments; it is: for each 2 inches increase in diameter from 8 to 30 inches, add 0.01 to 2.93 to obtain the required value of K . Above 30 inches the rate of increase will probably be less, and values of 3.06, 3.09 and 3.12 for 40, 50 and 60 inches diameter respectively are not likely to be more than 2 per cent. in error.

The curve connecting the logarithms of h_v and h_w previously referred to was found to be quite straight so long as the head over the weir was less than a certain fraction of its diameter. This fraction, as a result of many experiments on the 6-inch, 9-inch and 12-inch weirs, proved to be $\frac{1}{3}$. Quite possibly the law would hold a little beyond this, but it is certainly true up to this limit. The two typical curves, *Figs. 6 and 7*, show not only that the results give a straight line, but also the general accuracy of the observations.

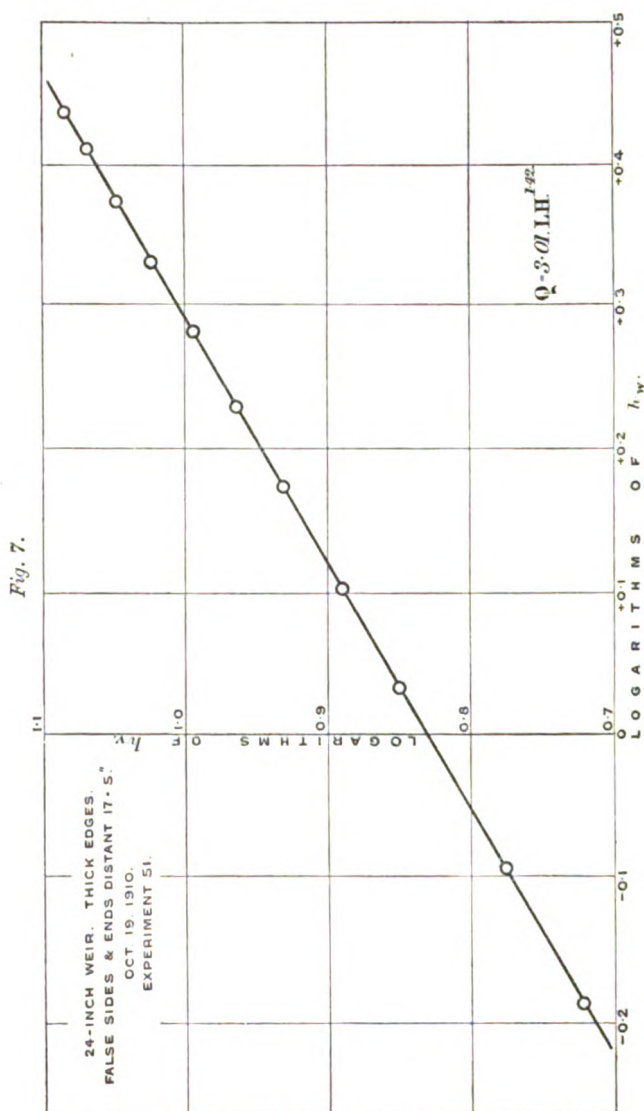
Comparison with Francis's Formula.—The results throughout have been expressed so that a direct comparison with the generally accepted Francis's formula for weir flow is possible. This formula for a sharp-edged weir without end contractions may be written $Q = 3.33 L H^{1.50}$. The limiting value of the index obtained in this

research for normal flow over a weir is 1.42 as against Francis's 1.50. This is the result of many observations, and there is little



reason to doubt its accuracy. This difference is striking and in the Author's opinion makes it doubtful whether the use of the higher

value in formulas expressing the law of flow over weirs is justifiable. If the many existing formulas are recast, using the lower value, it



is probable that certain troubles with the necessary constants will be eliminated. This conclusion is confirmed by the results given

in an interesting Paper¹ by Dr. J. H. T. Tudsbery, M. Inst. C.E., published about 25 years ago. He noticed how the coefficient varied with various experimenters under different ratios of L/H , and by comparison of known discharges over 3-foot and 10-foot notches and their corresponding heads he deduced a formula, $Q = 3.43 (L - 0.34) H^{1.43}$, giving the flow over notches not less than 3 feet long and proved its accuracy to within 2 per cent. by observations on the flow over an 8-foot notch.

The coefficients obtained in this research are considerably less than either Francis's or Tudsbery's, and clearly show that for circular weirs such as are likely to be used in practice, it would be quite out of the question to use either. In circular weirs there is strictly no end contraction, but there is a certain contractive effect extending over the whole circumference, and the experiments indicate that this effect, regarded as causing a deviation from the straight-weir law, decreases as the diameter increases. That it does not depend in any way on the head over the weir is evident from the fact that a straight line is obtained in every case when the logarithms of the head and quantity are plotted together. The limiting value of the coefficient should agree with that for a straight weir, and if Tudsbery's formula be accepted as a guide it will probably be about 3.40 to 3.45.

The formulas given are true for heads up to one-fifth of the diameter of the weir. The minimum head is governed principally by the thickness of the lip, and to obtain the maximum range for a weir a thin ($\frac{1}{32}$ -inch) brass lip is recommended; flows down to 0.4 inch may be thus obtained. It is advisable also to fit an air-pipe of a suitable size to the weir. If this be not done, the air which will be trapped beneath the jet will probably be gradually removed and the jet induced to cling to the interior of the pipe; for this state of affairs the laws deduced above will not hold.

From a careful consideration of the results, the minimum chamber appears to be a square having a side three times the diameter of the weir in length with a minimum size of 3 feet square. Probably for weirs greater than 30 inches in diameter the relative size of the chamber may be somewhat reduced without seriously affecting the law of flow.

The width of the channel of approach should not be less than twice the diameter of the weir unless some system of baffles is introduced to prevent the water short-circuiting.

¹ "The Gauging of Flowing Water." Minutes of Proceedings Inst. C.E., vol. lxxx, p. 318.

To remove the effect of velocity of approach, the depth of water in the chamber below the lip of the weir was in no case less than five times the maximum flow over it, and this ratio is recommended for practical use. Finally, as the results obtained by measuring the head "opposite" have been used in deducing the laws, it will be more satisfactory to do the same in practice.

Conclusion.—The experiments were carried out at the University of Liverpool during the year 1909–10 by the Author, whilst holding a Research Scholarship. Owing to a temporary lack of accommodation elsewhere, Professor Marchant very kindly allowed the work to be carried out in the Laboratories of Applied Electricity.

The research was necessarily costly, and the Author would place on record his grateful appreciation of the substantial financial help afforded him by the Council of the University. He is indebted to Mr. Joseph Parry, M. Inst. C.E., Water Engineer of Liverpool, for the use of the pipes forming the weirs, and to Mr. J. A. Brodie, M.Eng., M. Inst. C.E., for supplying the concrete materials and lending numerous useful articles. Dr. J. H. Grindley and the members of the engineering staff, by their timely help and advice, greatly contributed to the successful completion of the experiments. The research was suggested by Dr. Herbert Lapworth, Assoc. M. Inst. C.E., and throughout the experiments the Author has been much encouraged by his kindly interest and criticism. The Author would also thank Messrs. H. H. Kay and T. E. N. Fargher, engineering students, who during several weeks of their summer vacation helped by taking a number of photographs and assisting with observations.

The Paper is accompanied by five sheets of drawings, from which the cuts in the text have been prepared, by several photographs, and by the following Appendix.

APPENDIX.

TABLE I.—12-INCH WEIR.

Experiment 37.

Observation Sheet.

Date.—Monday, 10th October, 1910.

Conditions.—All clear. Jet flow. Thick edge. Head measured opposite Sides and end distant 31·75 inches from weir.

Experi- ment.	V Notch.	Vernier.	Reading.	Weir.	Vernier.	Reading.	Experiment Ended.
1	5·759	0·760	..	7·513	0·510	..	11·20
2	6·303	0·306	..	7·674	0·672	..	11·35
3	6·743	0·749	0·743	7·812	0·812	0·812	11·54
4	7·356	0·359	0·360	8·024	0·020	0·021	12·13
5	7·926	0·927	..	8·230	0·230	..	12·33
6	8·500	0·499	..	8·461	0·462	..	12·54
7	9·085	0·076	0·079	8·714	0·710	0·712	1·15
8	9·658	0·656	..	8·975	0·975	..	1·31
9	10·261	0·261	..	9·268	0·267	..	1·51
10	10·770	0·769	..	9·530	0·530	..	2·16
11	11·209	0·207	..	9·771	0·774	..	2·37

TABLE II.—12-INCH WEIR.

Experiment 37.

Result Sheet.

Date.—Monday, 10th October, 1910.

Conditions.—All clear. Thick edge. Head measured opposite. Sides and end distant 31·75 inches from weir.

Zero of V notch $\left\{ \begin{array}{l} 1·755 \text{ 8th October} \\ 1·752 \text{ 12th } \text{,,} \end{array} \right\} = 1·754.$

Zero of weir $\left\{ \begin{array}{l} 6·881 \text{ 7th October} \\ 6·881 \text{ 11th } \text{,,} \end{array} \right\} = 6·881.$

Experi- ment.	Vernier Reading.	h_v	Log h_v	Vernier Reading.	h_w	Log h_w
		Inches.			Inches.	
1	5·760	4·006	0·6027	7·511	0·630	0·2007
2	6·305	4·551	0·6581	7·673	0·792	0·1013
3	6·746	4·992	0·6983	7·812	0·931	0·0311
4	7·359	5·605	0·7486	8·021	1·140	0·0569
5	7·927	6·173	0·7905	8·230	1·349	0·1300
6	8·500	6·746	0·8291	8·462	1·581	0·1990
7	9·080	7·326	0·8649	8·712	1·831	0·2627
8	9·657	7·903	0·8978	8·975	2·014	0·3209
9	10·261	8·507	0·9298	9·267	2·386	0·3777
10	10·770	9·016	0·9550	9·530	2·649	0·4231
11	11·208	9·454	0·9757	9·772	2·891	0·4611

Results.—1st setting of line $Q = 3·002 \text{ L H}^{1·424}.$
2nd " " " $Q = 2·962 \text{ L H}^{1·421}.$
Mean $Q = 2·980 \text{ L H}^{1·42}.$

TABLE III.—RESULTS OF EXPERIMENTS ON 6-INCH WEIR.

Exact diameter = 6.91 inches.

Conditions.	Distance in Inches.	Head measured at side.		Refer- ence No.	Head measured opposite.		Refer- ence No.
		K	m		K	m	
Thick edge. All clear .	74.0	2.68	1.39	1			
Thin " " " .	35.0	2.71	1.39	2	2.70	1.39	3
" " Passage .	62.5	2.62	1.38	4			
" " " .	53.0	2.67	1.38	5			
" " " .	43.5	2.75	1.40	6			
" " " .	40.0	2.66	1.39	7			
" " " .	30.0	2.66	1.39	8			
False sides and end .	26.5	2.79	1.41	9			
" " " .	22.5	2.71	1.39	10			
" " " .	17.5	2.65	1.38	11			
" " " .	14.5	2.75	1.40	12			

Law for this weir is $Q = 2.93 L H^{1.42}$.

TABLE IV.—RESULTS OF EXPERIMENTS ON 9-INCH WEIR.

Exact diameter = 10.08 inches.

Conditions.	Distance in Inches.	Head measured at side.		Refer- ence No.	Head measured opposite.		Refer- ence No.
		K	m		K	m	
Thick edge. All clear .	33.5	2.96	1.42	13			
Thin " " " .	74.0	2.96	1.43	14	3.06	1.44	24
" " Passage .	19.0	2.89	1.42	15	3.00	1.42	25
" " " .	15.0	2.96	1.42	16	2.84	1.40	26
" " False sides and end }	13.0	2.89	1.42	17	2.98	1.43	27
" " " " }	15.5	2.94	1.42	18	2.90	1.42	28
" " " " }	20.5	2.93	1.42	19	2.91	1.42	29
" " " " }	25.0	2.91	1.42	20	2.94	1.42	30
" " False sides .	13.0	2.93	1.42	21			
" " " " .	15.5	2.96	1.43	22			
" " " " .	20.5	2.94	1.42	23			

Law for this weir is $Q = 2.94 L H^{1.42}$.

TABLE V.—RESULTS OF EXPERIMENTS ON 12-INCH WEIR.

Exact diameter = 13.70 inches.

Conditions.	Distance in Inches.	Head measured at side.		Refer- ence No.	Head measured opposite.		Refer- ence No.
		K	m		K	m	
Thick edge. All clear . . {	32.0	2.99	1.42	31	2.98	1.42	37
{	74.0						
{	52.5	2.98	1.42	32	2.99	1.42	38
{	39.5	..	..	..	2.90	1.41	39
{	23.0	3.01	1.43	33	2.95	1.42	40
{	18.5	2.95	1.42	34	2.93	1.42	41
{	14.0	3.04	1.43	35	2.98	1.42	42
{	11.5	3.04	1.43	36	3.05	1.44	43

Law for this weir is $Q = 2.97 L H^{1.42}$.

TABLE VI.—RESULTS OF EXPERIMENTS ON 18-INCH WEIR.

Exact diameter = 19.40 inches.

Conditions.	Distance in Inches.	Head measured at side.		Refer- ence No.	Head measured opposite.		Refer- ence No.
		K	m		K	m	
Thick edge. All clear . . {	29.0	..	..	..	3.01	1.42	44
{	74.0						
{	52.5	..	..	..	2.91	1.41	45
{	39.5	..	..	..	2.88	1.41	46
{	21.0	3.06	1.43	48	2.97	1.41	47 (a)
{					3.02	1.42	47 (b)

Law for this weir is $Q = 2.99 L H^{1.42}$.

TABLE VII.—RESULTS OF EXPERIMENTS ON 24-INCH WEIR.

Exact diameter = 25.90 inches.

Conditions.	Distance in Inches.	Head measured at side.		Refer- ence No.	Head measured opposite.		Refer- ence No.
		K	m		K	m	
Thick edge. All clear . .	25.5	3.09	1.42	49	2.98	1.41	50
{	17.5	..	..	..	3.01	1.42	51

Law for this weir is $Q = 3.03 L H^{1.42}$.

(Paper No. 3898.)

"A Direct-Reading Instrument for Submarine-Cable and other Calculations."

By ROLLO APPLEYARD, M. Inst. C.E.

IN predetermining the speed of signalling through submarine cables, and the relationship between that speed and the cost of the conductor and of the dielectric of the cable, the principal term is $\log D/d$, where D is the diameter of the dielectric, and d is the effective diameter of the conductor. This term also appears when calculating the capacity constants and the dielectric-resistance constants of a given dielectric, from tests of the cable-core; and it enters into problems relating to the transmission of electrical energy through cables. For any selected pair of values of D and d , it is, of course, easy to find the ratio of their logarithms from tables; but for any selected value of $\log D/d$ there is an infinite number of pairs of values of D and d , and the labour of tabulating a selected list of them is considerable. It has been attempted, therefore, to devise a direct-reading instrument, the equivalent of a slide-rule, to indicate all values of this ratio, where D and d are both variables. It is found that, by appropriately marking the scales of such an instrument, it can be arranged to indicate, by a single setting, and for any selected pair of values of D and d , the following quantities:— $\log D/d$, the inductive capacity, the dielectric resistance, the conductor resistance, the self-inductance of an "unloaded" loop, the weight of the conductor, and approximately the weight of the dielectric, per unit length of cable, of given materials under standard conditions.

The instrument (Figs. 1, 2 and 3, Plate 9) is circular and depends upon the use of a logarithmic spiral, the pole of which is at the centre of the circle. This circle is divided into degrees, and the two radial arms, each of which is free to turn about the centre independently of the other, can thus be set to any required angle. Each radial arm is provided with a scale of equal divisions, and the zero marks of these scales are always at the pole of the spiral, i.e., at the pivot of the arms. One radial arm can be allotted to D , and the

other to d , and they can be set to intersect the spiral to correspond with a given pair of values of D and d , as read upon their respective scales. If the angle between the radial arms be denoted by $(\theta_1 - \theta_2)$, for direct-reading the shape of the spiral must be such that for all pairs of values of D and d , this angle $(\theta_1 - \theta_2)$ must be proportional to $\log D/d$.

Constants of the Spiral.—Let the equation to the spiral be written in the form—

$$\theta = M \log_{10} r - N \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where r is the radius-vector, and M and N are constants to be determined. If $D_1 d_1$ are two values of r , corresponding with θ_1 and θ_2 respectively, then—

$$\theta_1 - \theta_2 = M \log_{10} \frac{D_1}{d_1} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Now, if ambiguous readings are to be avoided, the spiral must consist of only one convolution. Consequently, the greatest value that θ can have is 360° , and the least is 0° .

For accuracy readings near the pole of the spiral must be rejected, therefore an inferior limit must be assigned to r , which in the case under consideration will be determined by the least value of d , expressed in any convenient unit that enters into telegraph-cable work. Let this value of d be $d_{\min}$. Similarly, if the instrument is to be kept within any required limits of size, the length of the radial scale will be determined by $D_{\max}$, the greatest diameter of dielectric, expressed in the given unit. In order, therefore, to arrive at the best conditions, taking the general case, the following values must be written in equations (1) and (2)—

$$\theta_1 = \theta_{\max} = 360^\circ$$

$$\theta_2 = \theta_{\min} = 0^\circ$$

$$D_1 = D_{\max}$$

$$d_1 = d_{\min}$$

Whence the best values for the constants are found to be—

$$M = \frac{360}{\log_{10} \frac{D_{\max}}{d_{\min}}} \quad N = \frac{360 \log_{10} d_{\min}}{\log_{10} \frac{D_{\max}}{d_{\min}}}$$

In submarine-cable work it is usual to express diameters in mils or thousandths of an inch, and the minimum diameter of the con-

ductor is about 32 mils, and the maximum diameter of the dielectric is about 500 mils. Substituting these values of $D_{\max}$ and $d_{\min}$

$$M = 301.6 \quad N = 453.9.$$

The best conditions, in the case of submarine-cable work, will therefore be sufficiently represented by taking $M = 300$, and $N = 450$, so that equation (1) becomes—

$$\theta = 300 \log_{10} r - 450 \quad . \quad . \quad . \quad (3)$$

where r can be given successive values from about 32 to about 500. In this case, since $D_{\max} = 500$, each radial arm is divided into 500 equal parts, and D_1 and d_1 respectively can be read thereon in mils. With the spiral represented by equation (3), if one of the arms is set at 0° on the divided circle, the spiral will intersect the radial scale at 31.6 divisions; and while the arm is rotated from 0° to 360° , its point of intersection with the spiral will move along the radial scale to 501.2 divisions. The range, therefore, is satisfactory, and the whole of the spiral is used to the best advantage. It should be observed that the equation giving the best conditions is independent of the size of the selected spiral. Nevertheless, the accuracy depends upon having a fairly open scale for the radial arms, and they should therefore be as long as is consistent with convenience of handling. The length chosen for the maximum radius is immaterial, but it is convenient to select it so that it can be easily graduated into 500 equal parts by means of a scale of subdivided inches or millimetres. After this graduation, the divisions on the radial scale can be regarded as representing mils, millimetres, inches, or any units whatever; but for cable work they will generally be read as mils.

Reading Log D/d.—Having decided to make $M = 300$, equation (2) becomes—

$$\log_{10} \frac{D_1}{d_1} = \frac{\theta_1 - \theta_2}{300}$$

If, therefore, a circle is drawn, concentric with the circular scale of degrees, and so marked that each division upon it coincident with a given value of θ° reads $\frac{\theta^\circ}{300}$, the scale on this circle will give a direct-reading of $\log D_1/d_1$. The angle corresponding with the setting of the radial arm for D_1 is θ_1 , and the angle corresponding with the setting for d_1 is θ_2 . Hence, when the radial arms are set on the spiral in positions corresponding with D_1 and d_1 respectively, the angle between them is $(\theta_1 - \theta_2)$. There are several ways in which

this angle may be measured. A convenient method is to clamp the two radial arms together, and then to bring one of them to 0° . The other arm then stands at $(\theta_1 - \theta_2)$ on the circle of degrees, and at $\frac{(\theta_1 - \theta_2)}{300} = \log_{10} \frac{D_1}{d_1}$, on the concentric circle of logarithms.

In Fig. 1 the radial arm giving the diameter D of the dielectric is shown intersecting the spiral at 403, and the reading on the graduated circle is 331.6; the radial arm giving the diameter d of the conductor intersects the spiral at 250, and the reading on the circle is 269.4° . Hence $\theta_1 - \theta_2 = 331.6 - 269.4 = 62.2$, and against this angle the value of $\log D/d$ is found to be 0.207.

Capacity and Dielectric Resistance.—If definite values are assigned to the dielectric material, the instrument can easily be graduated to read directly in microfarads and megohms for all values of D and d . For example, let K be the capacity in microfarads, and let R be the resistance in megohms of the dielectric as measured between the opposed faces of a cube, of which the side is 1 nautical mile; and let c and z be the capacity and dielectric resistance, in microfarads and megohms, respectively, of a nautical mile of cable made with this dielectric. Then, if the effective diameter of the conductor of the cable is d , and if the diameter of the dielectric is D , the relationship is—

$$\frac{c \log_{10} \frac{D}{d}}{2.728} = K$$

and

$$\frac{2.728 z}{\log_{10} \frac{D}{d}} = R$$

If K and R are regarded as constants, the values of c and z can be written down for all values of $\log D/d$, and they can be marked upon concentric circles at the corresponding positions of $\log D/d$. For gutta-percha, K and R vary according to the quality and treatment of the material, but as an example it may be assumed that $K = 0.058$, and $R = 1,500$. In this case—

$$c = \frac{0.1582}{\log_{10} \frac{D}{d}} \text{ microfarads}$$

and

$$z = 549.8 \log_{10} \frac{D}{d} \text{ megohms.}$$

Weight of Conductor and Weight of Dielectric.—Against every main division of the radial scale from which values of d are read,
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can be written the corresponding weight, w , in pounds of the conductor per nautical mile. Similarly, against every division of the radial scale from which values of D are read, can be written the corresponding weight in pounds per nautical mile of a solid cylinder of dielectric D mils in diameter. The actual weight of the dielectric in the cable can then be obtained by subtraction as soon as d is known.

Secondary Spirals.—In cable-work, owing to the doubt that always exists regarding the true value of the effective diameter of a strand, $\log D/d$ is rarely required more accurately than within 1 per cent. It is somewhat disappointing, however, to find that even when such an instrument is constructed in accordance with the best conditions, this ratio cannot always be read with certainty to within the required 1 per cent. An examination of equations (1) and (2), however, reveals a complete solution of the difficulty, and this solution is of wide application to engineering problems of the same class.

The problem is to find the values of M and N that will enable $\log D_1/d_1$ to be read off the circular scale of degrees, or off a circular scale of equiangular divisions other than degrees, with maximum precision.

By differentiating equation (1)—

$$\frac{d\theta}{dr} = \frac{M}{r} \log_{10} e = 0.4343 \frac{M}{r}$$

In submarine cable-work as already stated, r varies from about 32 to 500 mils. Putting $M = 300$ in this equation—

$$\frac{d\theta}{dr} = \frac{130.29}{r}$$

And substituting for r first 32 and then 500, it will be seen that $\frac{d\theta}{dr}$ varies from 4.07 to 0.261. A small error in reading the radial scale near the thirty-second division corresponds, therefore, to a change $\frac{4.07}{0.261} = 15.6$ times greater in θ than does the same error in the radial scale-reading near the five-hundredth division. Readings very near the pole of the spiral, therefore, must be rejected. But, on the other hand, if readings begin at too great a distance from the pole the instrument becomes unwieldy.

If each subdivision of the radial scales is supposed to be made n times greater, where n is constant, the point of intersection between the scale and the spiral for a given scale-reading will be n times the original distance from the centre of the divided circle,

and in order to move from the original to the new position, the arm must be rotated through an angle $(\theta_1 - \theta_n)$. But since n is constant, $\log_{10} n$ is constant; and as the angle of rotation is proportional to $\log_{10} n$, it is clear that $(\theta_1 - \theta_n)$ is constant. This means that in order to magnify all the subdivisions of the radial scales n times, every point on the spiral has simply to be rotated through the same constant angle $(\theta_1 - \theta_n)$ about the centre of the divided circle. The new spiral is, therefore, the same as the original spiral, but it is moved round the pole through an angle of $(\theta_1 - \theta_n)$ degrees. Incidentally it may be observed that only one template is necessary for drawing any number of such spirals. Writing n for $\log D/d$ in equation (2)—

$$\theta_1 - \theta_n = M \log_{10} n$$

For example, if $n = 5$, and $M = 300$ —

$$\theta_1 - \theta_5 = 300 \times 0.69897 = 209.69 \text{ degrees.}$$

Consequently, if on the original instrument there is drawn a new spiral in addition to the first one, but differing from it by 209.69 degrees in phase, the divisions on the radial arms can be made five times as large as before. It is convenient to extend every fifth division of the original scale to form a new scale; each such division on the new scale can then be marked to read five times the reading on the original scale. If the original scale reads up to 500 mils, the new scale will read only up to 100 mils. The new scale can be very narrow, and it may conveniently be arranged along the bevel edge of the radial arm, or it may be formed by renumbering the original scale. A radial arm provided with such scales can then be read where it intersects either or both of the spirals, and the readings are in agreement for both spirals, for all angles. Such a scale gives $\log D/d$ accurately to within about $\frac{1}{2}$ per cent.

The Table in the Appendix gives the values of $(300 \log r - 450)$ for a sufficient number of values of r to enable the spirals to be plotted. If a radial arm of any selected length whatever is divided into 500 equal parts, and also into 100 parts, with a scale marked to each set of divisions, the figures given in this Table enable the two curves to be drawn, corresponding with these two scales. The same Table is also available, without alteration or addition, for drawing a curve whatever the number of equal parts into which the selected length of radial arm is divided.

It should be observed that one great advantage of the instrument

is that when the pair of radial arms is clamped together at any angle corresponding with say D_1 and d_1 , they can be rotated together about the pivot so as to give every possible pair of values, D_n and d_n , such that $\log \frac{D_n}{d_n} = \log \frac{D_1}{d_1}$, and the best pair of values of D and d can thus be chosen to accord with mechanical and other conditions.

By the means just described, the logarithmic spiral is rendered available for a number of practical cases that arise in physics and in engineering, where a result is required to be read off directly, rapidly, and without any calculation; for it is obvious that this new principle of secondary spirals can be extended to give a degree of precision formerly unattainable. It may be useful to add the following characteristic examples of problems to which such instruments could be applied.

(1) In acoustics the spiral can be used to connect frequency with "interval." The equation is—

$$\theta - \theta_1 = 2\pi \frac{\log r - \log r_1}{\log 2},$$

where r represents the number of vibrations per second, and $(\theta - \theta_1)$ is a graphical representation of n/n_1 , the ratio of the two frequencies—which is generally, but somewhat inaccurately, termed the "interval."

(2) All "damping" problems involving a logarithmic decrement.

(3) Most problems depending upon the "compound interest" law, or temperature coefficients.

(4) The work done when a gas expands according to Boyle's law. The work (W) in foot-lb. is given by—

$$W = 2.3 R T \log_{10} \frac{v}{v_0},$$

where v_0 and v are the initial and final volumes respectively, T the absolute temperature, and R the gas-constant in foot-lbs.

(5) In modern cable-work inductance has to be taken into account. For two straight parallel conductors, each of radius a , separated by a distance b between centres, so as to form a "loop," the inductance L in centimetres is—

$$L_{\text{cms}} = \left(1 + 4 \log \frac{b}{a}\right) \text{ per centimetre of loop.}$$

Reducing this to practical units—

$$L = \left(0.1856 + 1.7094 \log_{10} \frac{b}{a}\right) \text{ millihenries per nautical mile of loop.}$$

In the case of a twin-core cable, where the two cores are in contact, b is the diameter of the dielectric, and a is the radius of the conductor. Hence replacing b and a by D and $d/2$ respectively, these being the usual symbols, the last equation takes the form—

$$L = \left(0.7002 + 1.7094 \log_{10} \frac{D}{d} \right) \text{ millihenries per nautical miles of loop.}$$

By marking on the circular scale, against each value of $\log D/d$, the corresponding value of $(0.7002 + 1.7094 \log D/d)$, the instrument could easily be used to calculate this inductance. The setting that gave $\log D/d$ would then give simultaneously the value of the inductance for any selected values of D and d .

The Paper is accompanied by three diagrams, from which Plate 9 has been prepared ; also by the following Appendix.

[APPENDIX.

APPENDIX.

TABLE OF VALUES FOR PLOTTING SPIRALS.

r	$300 \log r - 450.$	r	$300 \log r - 450.$	r	$300 \log r - 450.$	r	$300 \log r - 450.$
20	-59.7	65	93.9	145	198.4	270	279.4
21	-53.4	67	97.8	147	200.2	280	284.1
22	-47.3	70	103.5	150	202.8	290	288.7
23	-41.5	72	107.2	152	204.6	300	293.1
24	-36.0	75	112.5	155	207.1	310	297.4
25	-30.6	77	115.9	157	208.8	320	301.5
26	-25.5	80	120.9	160	211.2	330	305.5
27	-20.6	82	124.1	162	212.9	340	309.4
28	-15.9	85	128.8	165	215.2	350	313.2
29	-11.3	87	131.9	167	216.8	360	316.9
30	- 6.9	90	136.2	170	219.1	370	320.5
31	- 2.6	92	139.1	172	220.7	380	323.9
31.62	0.0	95	143.3	175	222.9	390	327.3
32	1.5	97	146.0	177	224.4	400	330.6
33	5.6	100	150.0	180	226.6	410	333.8
34	9.4	102	152.6	182	228.0	420	337.0
35	13.2	105	156.4	185	230.2	430	340.0
36	16.9	107	158.8	187	231.6	440	343.0
37	20.5	110	162.4	190	233.5	450	346.0
38	23.9	112	164.8	192	235.0	460	348.8
39	27.3	115	168.2	195	237.0	470	351.6
40	30.6	117	170.5	197	238.3	480	354.4
41	33.8	120	173.7	200	240.3	490	357.1
42	37.0	122	175.9	205	243.5	500	359.7
45	45.9	125	179.1	210	246.7	510	362.3
47	51.6	127	181.1	215	249.7	520	364.8
50	59.7	130	184.2	220	252.7	530	367.3
52	64.8	132	186.2	230	258.5	540	369.7
55	72.1	135	189.1	240	264.1	550	372.1
57	76.8	137	191.0	250	269.4	560	374.5
60	83.4	140	193.8	260	274.5	570	376.8
62	87.7	142	195.7				

(*Paper No. 3912.*)

“Tramcar-Meters, and the Economical Results of their Adoption on a Tramway System.”

By ALFRED SHARMAN GILES, M. Inst. C.E.

IN addition to the meters that are fixed in the power-station to measure the output to any one section of a tramway system, tramway corporations and companies have long felt the want of detailed measurements of the current consumed by individual cars. As each tramcar may consume energy equal to 40,000 kilowatt-hours per annum, of a value of £150 to £200, a method of measuring this consumption on each car separately must tend to the more economical use of power; and the systematic readings will indicate whether the car-equipment is in good and efficient order.

TYPES OF METER.

The early forms of tramcar- or traction-meters, as they are now called, were not satisfactory. Being very delicate, and for the most part of the ordinary stationary type with some modifications, their accuracy was but short-lived when they were subjected to excessive shocks and continuous vibration. During the last few years, however, meters have been devised which counteract the troubles due to vibration, the moving parts being so suspended and supported as to avoid the transmission of the vibration to this part of the mechanism; and at the present time reliable commercial instruments can be purchased at a moderate price.

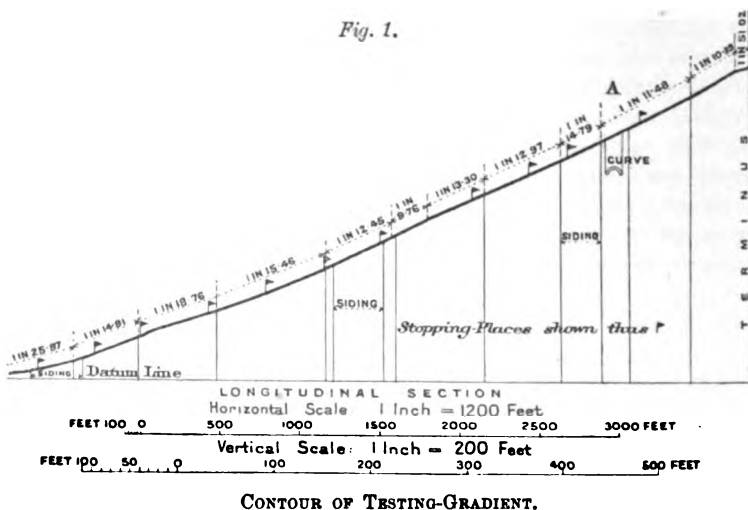
There are several types of traction meter:—the ampere-hour meter, the time-meter, and the recording wattmeter.

There are two well-known makes of ampere-hour meters, designed for tramcars; their chief feature being that the revolving mechanism floats in a mercury bath caused to rotate by the current passing; by this means the bottom bearing is relieved of the weight of the moving parts, and no hammering of the armature-spindle takes place. On a system where the mean voltage of supply is nearly constant, ampere-hour meters have given much satisfaction;

but as the dials of the instruments register units or kilowatt-hours at, say, 500 volts line-pressure, it will be seen that the readings are not accurate on a system where the line-pressure fluctuates about 10 to 15 per cent.

Time-meters are devised to record the number of minutes current is being used by the car. Some of these instruments have recording mechanism registering the number of minutes during which the motors are in series or in parallel. The records, however, are of only partial use, for these instruments do not take account of the variable current, which is excessive if the controller is advanced too quickly in starting or during acceleration.

Fig. 1.



The recording wattmeter measures the actual energy consumed, and the readings are accurate over a wide range of voltage.

The particular type of instrument used on all cars in Cape Town is especially designed to withstand vibration, and has proved itself fully capable of meeting all the requirements of the situation. The difficulty of the bottom bearing is ingeniously overcome; the lower pivot, instead of being solid with the armature-spindle, is made of highly tempered steel and takes the weight of the armature through a spring. Its jewel is also spring-supported, so that any vibration, which under normal circumstances might produce chattering, causes the jewel and the pivot-point to move up and down as one piece. All vibration is thus taken up by these upper and lower springs, and no hammering of the jewel is possible.

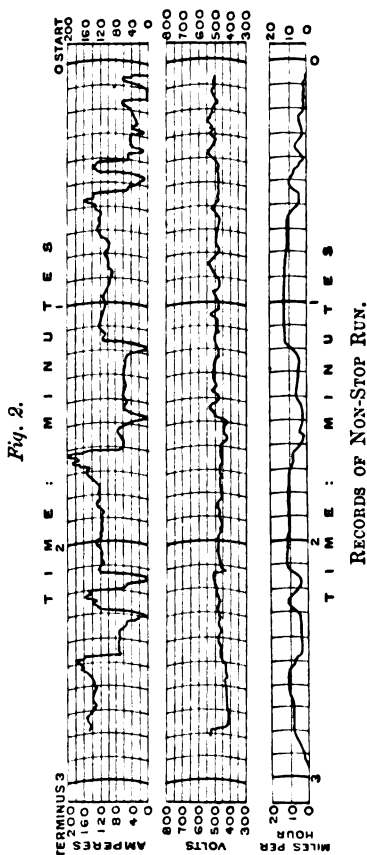
USE OF THE METERS.

From records taken over the tramway system in Cape Town before fitting all the cars with meters, it was ascertained that the same car, driven under exactly similar conditions over the same route, showed a difference of 40 per cent. in the quantity of electrical energy used by one motorman and another. In the Author's opinion, this difference can only be accounted for by ignorance and carelessness.

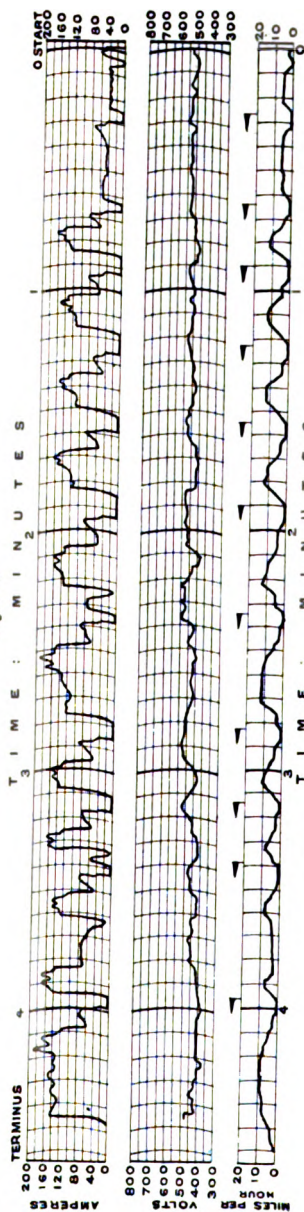
The electrical energy absorbed in the propulsion of tramcars is one of the most important items of expenditure in a tramway business. Not only this, but it has a direct effect on other large expenditure, namely, the life of the electrical and mechanical equipment of the cars and the wear and tear of the permanent way. In order to illustrate graphically the waste of electrical energy which may take place, some curves taken on a portion of a tramcar-route in Cape Town are reproduced. In a distance of 4,383 feet the line rises 306 feet, so that the average gradient is 1 in 14·3. The contour of the hill, as will be seen from *Fig. 1*, varies considerably throughout its length, the steepest gradient being 1 in 9·76 and the minimum gradient 1 in 25·97.

The weight of the test-car was 22,174 lbs., and, together with motormen and observers, was reckoned for easy calculation as 9·9 tons. It was fitted with four recording instruments, namely, wattmeter, ammeter, voltmeter and speed-indicator, the last three being specially arranged to pass 5 inches of chart per minute.

Observations.—The car was driven up this hill without a stop, slowing down only on entering and leaving the three passing-sidings

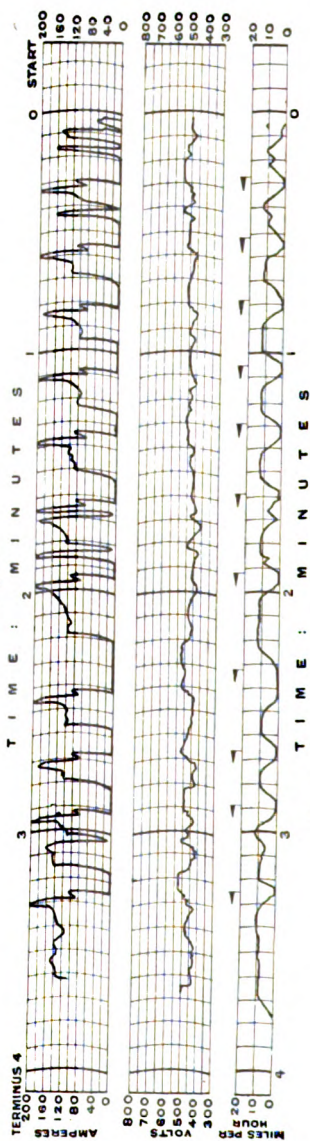


Figs. 3.



RECORDS OF STOPPING RUN WITH CORRECT STARTING.

Figs. 4.



RECORDS OF STOPPING RUN WITH FAULTY STARTING.

and the 37-foot curve marked at A in *Fig. 1*. The energy used was 3.75 kilowatt-hours, and the speed-curve, ammeter and voltmeter-charts under these conditions are shown in *Fig. 2*.

To illustrate the effect of passenger-stops, the car was then driven up the hill stopping at the fixed halting-places shown in *Fig. 1*. by flags, each stop being made for 5 seconds. The energy consumed was then 4.25 kilowatt-hours, and the records obtained under these conditions are shown in *Fig. 3*. These show the results of normal acceleration, and in *Fig. 4* is shown the effect of wrong or too rapid movement of the controller.

Fig. 5.

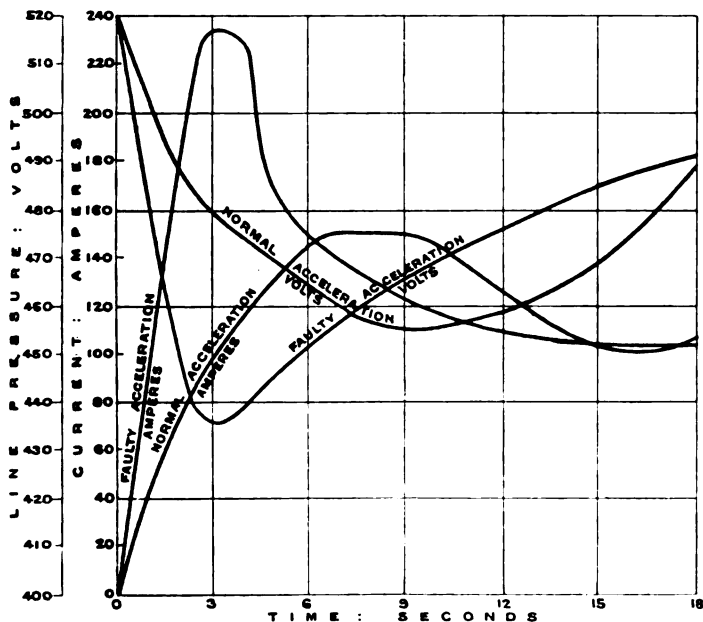


DIAGRAM OF CORRECT AND FAULTY STARTING: AVERAGED FROM FIGS. 3 AND 4.

It will be interesting to compare the results obtained from *Figs. 2* and *3*. There are eleven stopping-places on this route, and the energy used by the car on the non-stop run was 3.75 kilowatt-hours, while on the run including stops it reached 4.25 kilowatt-hours. The stopping therefore accounts for 11.76 per cent. of the total energy required to run the car up this gradient.

The effect of faulty starting and acceleration is shown diagrammatically in *Fig. 5*, which has been plotted from the curves

of *Figs. 3 and 4*; the actual calculations over a measured distance being as follows:—

	Faulty Start.	Correct Start.
Average power used HP.	84·3	74·1
Average loss in power HP.	10·2	..
Consumption Watt-hours	314	276
Energy wasted Per cent.	14	..
Time to bring controller to full parallel . Seconds	4	9
Maximum current Amperes	232	150
Pressure-drop during acceleration . . . Volts	85	65

It will be seen, therefore, that 14 per cent. of the energy required for starting can be wasted by faulty acceleration.

Energy may be wasted when approaching a stopping-place, for the acme of tramcar-driving being to use the stored energy in the car as far as possible, application of the brakes is only so much waste of that storage. A skilful man, therefore, in exercising correct judgment of speed and distance will waste less than one who "pushes" his car close up to the stopping-place, and then puts on the brakes hard.

RESULTS OBTAINED ON SYSTEM.

The effect of the personal element in tramcar-driving was illustrated very well from some results, obtained over the Wynberg section of the tramways. The energy consumed was averaged over 1 week for ten different motormen, when the figures ranged from a minimum of 1·2 to a maximum of 1·67 kilowatt-hour per car-mile.

About 1½ year ago, in order to encourage economical driving, a bonus system was instituted for the motormen, and met with some success, though there is room for still further improvement. For each section of the tramway system, a reasonable kilowatt-hour figure per car-mile is fixed, and all men averaging this figure during the week receive a bonus of 2s. 6d. A careful log of the meter-readings and mileage run per day is kept, together with the name of the motorman working the car. Two or more men may drive the car during the day; the figures, mileage and meter-readings are kept distinct, and at the end of the week are totalled up in the power-readings book. A list is posted each week showing the results of each man.

From what has been said already in connection with starting and stopping, a certain quantity of wasted energy can be accounted for, but beyond this the very great difference between one man and another can only be put down to carelessness, inattention, or ignorance.

In Cape Town the fixing of meters to all cars has had a marked effect on the power generated and the maintenance-costs of the equipments. To take a few items, the average lives of various parts were increased as follows:—

	Before Meters were adopted.	After Meters were installed.
	Miles.	Miles.
Average life of armature-bearings	23,450	33,766
„ „ „ motor-pinions	27,883	31,916
„ „ „ axle-bearings	58,750	92,333
„ „ „ car-wheels	45,833	55,833
Expenditure on brake-shoes	£332	£196
Average power used over a period of 12 months) Kilowatt-hour per car-mile)	1·814	1·546

There was thus a saving of 41 per cent. in the expenditure on brakes and a saving of 14·77 per cent. in the average power used.

In addition, such items as the armature-windings and field-coils of the motors are not subject to such heavy current and high temperatures, with the result that their life is considerably prolonged; and the saving in maintenance-cost of these items in the present case is approximately 33 per cent.

The wear and tear of permanent way with more gradual starting and stopping is not so great, for it is to be remembered that acceleration and retardation must be imparted between the periphery of the wheels and the rails, and, if this exceeds normal adhesion, the result is excessive grinding away of both wheels and rails.

EFFICIENCY-CALCULATIONS.

Some interesting calculations can be made from the results obtained on the test-hill, the gradients of which are shown in *Fig. 1*.

From the meter-readings the average electrical power taken during the non-stop trial-run was 54·83 HP. The theoretical power required to overcome rolling-resistance, gradient- and air-

resistance was calculated to average 44·81 HP., at 9 miles per hour.¹ The overall efficiency, therefore, is 81·72 per cent., the losses being accounted for in the motors and gears and slip of wheels.

These losses are very moderate considering that the motors are of a type (G.E. 1,000) designed about 16 years ago. For these series-wound traction-motors, at what is called half-speed, i.e., the motors on the cars being in full series at 250 volts, the efficiency would be about 75·3 per cent. In full parallel at 500 volts the efficiency would be about 81·5 per cent., including gear-losses.

The foregoing results were obtained under non-stop conditions for the hill-climb: the overall efficiency including stops would be less, and difficult to calculate.

The Paper is accompanied by four sheets of diagrams, from which the Figures in the text have been prepared.

¹ In these calculations the rolling-resistance was taken at 30 lbs. per ton, and the air-resistance allowed for was $0\cdot0017 V^2$ lbs. per square foot, V being the speed in miles per hour.

(*Students' Paper No. 647.*)

"Reconstruction of the Road-Bridge over the River Tame
at Lea Marston."¹

By VICTOR MICHAEL BARRINGTON-WARD, B.Sc., Stud. Inst. C.E.

[*Abridged.*]

THE bridge, the reconstruction of which is the subject of this Paper, is on the main road between Whitacre and Birmingham, and lies on the west side of the Midland Railway's new express line between Kingsbury and Water Orton. The widening of the existing bridge or the construction of a new one seems to have been necessary for some years, for not only was the structure in a dangerous state, a large crack having shown itself in one abutment, and the parapets overhanging $3\frac{1}{2}$ inches and 1 inch respectively, but the width (13 feet) of the roadway had become inadequate for the traffic, in addition to which the approach on the Lea Marston side was on a curve with a falling gradient towards the bridge.

The plans for the Midland Railway's Kingsbury and Water Orton branch, the Act for which was obtained in 1906, provided for the railway passing under the road, which was to be diverted with a raised approach of 1 in 26, at a point about 8 chains east of the bridge in question, and crossing the River Tame at a point about 150 yards farther up than the road-bridge. As the railway itself at this point was on an embankment, a somewhat lofty bridge was required, which in turn entailed a long approach. On the Lea Marston side this commenced at the bridge under consideration. The new approach was 25 feet wide between fences, consisting of an 18-foot road and two 3-feet 6-inch verges; this therefore caused a dangerous bottle-neck, the roadway tapering down from 18 feet to 13 feet with no footpath accommodation at all on the bridge. The small spans of the arches and the existence

¹ This Paper was read and discussed at a Meeting of Students in Birmingham on the 16th December, 1909.

of piers in the river were also a great obstacle in flood-time, as trees and other debris coming down got jammed broadside on between the piers, damming up the water.

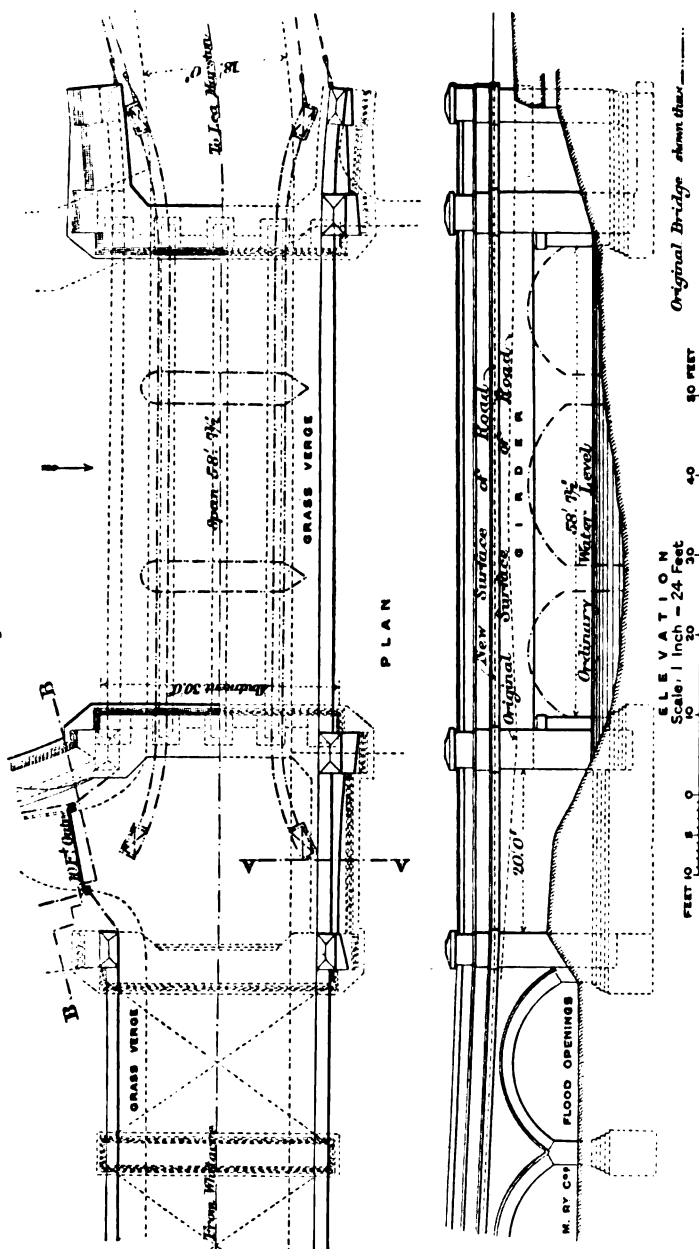
This district is particularly liable to floods, the river having to deal with the storm-water of Birmingham besides its own supply; and even after a small storm the river will rise 3 or 4 feet. During the construction of these works, it was found that it usually reached its maximum flow about 6 hours after the actual storm, while another feature was the fact that the water rose 5 or 6 inches at night, owing no doubt to the impounding of water at the dams of the mills lower down the river at Kingsbury, etc. When, therefore, the new approach for the road-bridge over the railway was planned, provision had to be made for dealing with the flood-water; consequently seven 20-foot flood-openings were built under the approach alongside the old bridge, the extent of the openings, together with the openings of the river bridge, being made equal to that of the openings under the railway-viaduct over the river higher up.

Three schemes were proposed to widen the bridge, but it was finally decided that a new single-span bridge should be built. Two plans were prepared for this; one was for two main girders with trough flooring between, and the other for five main girders with brick jack-arches between. The latter plan was adopted as being more satisfactory and economical to maintain, whilst involving a very small extra initial outlay.

The clear span of the new bridge (*Figs. 1*) is 58 feet 7½ inches, the same as the distance between abutments of the old bridge, the distance apart of the main girders being 6 feet 7½ inches and the distance between parapets being 25 feet. The new work was commenced on the 14th June, 1909. Both banks of the river at this point are covered with trees, and any which were thought might either endanger the new bridge by being blown on to it, or cause any movement in the foundations, through the swaying of the upper portion of the tree in a wind, and thus cause movement in the roots, were cut down. The crack in the brickwork of the old bridge already alluded to may be perhaps put down to the latter reason, as an old oak was growing only a few yards from the abutment in question.

A temporary bridge for pedestrians was first put over the river, and the road was then closed for vehicular traffic. The old bridge was then demolished, the parapets and road-metalling being first removed and then the filling over the arches. Stages were erected underneath the arches, and two of the three rings of brickwork were removed; the innermost ring, however, was dropped on to the stage beneath and then removed.

Figs. 1.



The pier on the Whitacre side was next removed, as owing to the encroachments made in the river by the works for the new abutments, it was considered advisable to keep the remaining waterway as clear as possible in view of the liability to floods. Excavation was first commenced for the Lea Marston abutment. On this side the depth of water was found to vary between 1 foot and 2 feet 6 inches at low water. A bag stank was employed to keep the water out, built on either side of the Lea Marston pier which was left in for this purpose, towards the bank. A bag stank consists of a series of bags, filled with earth, and laid one on top of another, and cemented together with mud and wet earth. The bags are of any size, but the best and handiest are about 3 feet long and 15 inches in diameter; in this particular case old sugar bags were bought from a jam factory at 2s. 6d. per dozen. They are most efficient when not filled too full, so that they may easily take up the shape of the bags below and alongside. The thickness of the stank depends, of course, on the depth of water, but in this case it was found that a thickness of 2 feet 6 inches easily kept out a depth of 5 feet of water. When the stank was done with, the bags were slit up with a knife, emptied of their earth and thrown away.

In the excavation for this abutment no trouble was met with, the marl was found about 10 feet down, and very little timbering was required. The foundations, however, were taken down 15 feet to a level below the deepest portion of the river in the immediate vicinity, in case a scour occurred at any time near this abutment.

While work was being carried out on this side of the river, pile-driving was proceeding in a cofferdam for the other abutment, and for the adjoining retaining wall, as owing to the nature of the ground and depth of water a bag stank was not feasible. The king piles were first driven 12 feet apart between centres, 12-foot piles being found sufficient except alongside the old abutment, where the water was deeper and 15-foot piles were required. A 7-cwt. monkey was used, the usual drop being from 3 to 6 feet. The piles were driven about a foot in the marl. During the first few blows the piles went in quickly, but when the marl was reached the progress was about $\frac{1}{2}$ inch to 1 inch a blow. It may be mentioned here that throughout the work everything was done by hand, it being considered more economical than bringing steam-plant for work of this description. Excavation on this side was extremely difficult compared with the other side; the ground was very wet and soft and close timbering by means of runners had to be resorted to. Water seemed to find its way in from all sides, but more especially from the land side, and two 4-inch Edson pumps were run night

and day to keep it out. This was probably caused by the water in the adjacent land percolating through the gravel stratum.

The marl was reached about 17 feet below the surface of the ground, and at a level of about 7 feet below the marl-level on the other side, the width across being about 58 feet; there was thus a dip of about 1 in 8. This however, was not unexpected, as when the foundations for the abutments and piers of the flood-openings alongside were being taken out, during the progress of the railway works, it was found that as the distance from the river increased the foundations had to be deeper and a greater quantity of water was encountered.

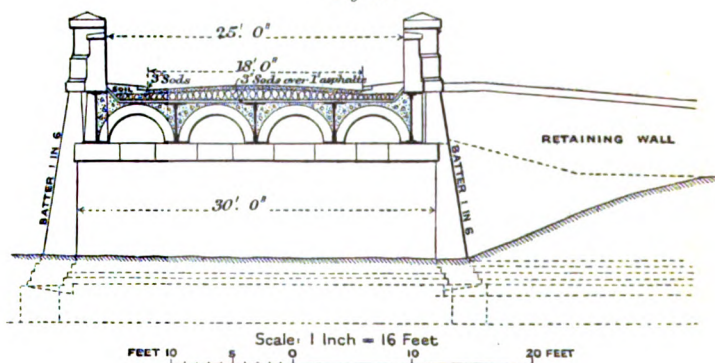
On completion of the excavation, cement concrete was put in. This was measured out in a banker in the proportion of 1 part of cement to 5 of good hard broken stone capable of passing through a $2\frac{1}{4}$ -inch ring, the interstices being filled up with sufficient good sharp sand. The mixture had, after being turned once, to be watered with a watering-can fitted with a rose and turned three times before being dropped into the bottom. The cement used had to conform with the British Standard specification for medium-setting Portland cement.

The bricks used were specified to be sound, hard, square, well burnt and of uniform size. For all face-work, Staffordshire brindled bricks were used, and no joint was allowed to exceed $\frac{1}{4}$ -inch. The lime mortar consisted of one part lime to two parts of sand; in the case of the face-work, black moulder's sand was used for colouring. As soon as the concrete was set, brickwork was commenced. The abutments were each 5 feet 3 inches thick, with footings 9 inches wide at the bottom reducing $2\frac{1}{4}$ inches every $6\frac{1}{2}$ inches. English bond was used throughout, while a diagonal course was put in every ninth course. Bull-nosed quoins were used. Lime mortar was used throughout, except for the girder-beds which were set in cement. The face-bricks were properly set by trowel, the inside courses being paved and "larried" in grout.

The steelwork was specified to be of a quality to withstand a tensile stress of not less than 26 tons per square inch and not more than $29\frac{1}{2}$ tons per square inch of section with an elongation in 8 inches of not less than 20 per cent. before breaking. The girders, five in number, consisting of three inside and two parapet girders each 64 feet long and weighing 14 and $12\frac{1}{4}$ tons each respectively, were conveyed to the site by road. The actual crossing of the stream was carried out by building a temporary bridge between the two abutments. Two timbers, each 84 feet long and 20 inches by 20 inches cross-sectional area, and weighing 6 or 7 tons, were used. Upon these, flat-bottomed rails were laid

and also up the road to where the girders had been lowered. A girder was then jacked up on to a couple of low-wheeled trolleys, and these were allowed to move slowly by gravity down the rails, being steadied by a winch on the up-hill side. No difficulty was experienced in the subsequent lowering into position, when once the girder was across the river. When each girder was in its correct position, four tie-bars on each flange, running from outside girder to outside girder, were riveted up to each to prevent spreading. The whole of the steelwork had to be painted, four coats being applied, one of these of red-lead had already been given in the shop after the girder had been well coated with boiled oil. The three coats on the site were of white-lead. The outside faces of the parapet girders were however not touched, the remaining three coats being left until the completion of the bridge. When

Fig. 2.



the paint was dry, a wall was built behind the girders on each abutment. This was carried up to the level of the top flange, the brickwork being built up to the end-plates of the girders, a space of $\frac{1}{4}$ inch however being left for expansion.

Jack-arching was then commenced. The first process consisted of laying a cement bed to cover the rivet-heads on the bottom flange, and upon this a course of bricks was laid, the drums which had been previously made were then set, resting on hangers hung from the bottom flanges of the two girders between which the arches were being turned. The hangers had a Z-shaped iron bar at each end, one of which acted as a bolt, so that when the drums had to be moved, it was only necessary to pull the bolt, when the hanger fell away and the drum dropped out, and could be set again elsewhere.

The arches consisted of two $4\frac{1}{2}$ -inch rings set in lime mortar, the inner of brindled, the outer of common red brick; a header of brindled brick, however, ran through as the "key." For the inner ring alternate courses of ordinary brindled brick $3\frac{1}{8}$ inches thick, and culvert blue brick, which tapered from $3\frac{1}{8}$ inches to $2\frac{1}{8}$ inches, were used, thus giving a smaller joint on the extrados than would have been the case if ordinary $3\frac{1}{8}$ -inch blue bricks had been used throughout. For the outer ring no special shaped bricks were used. The whole of the space between the arches and the level of the top of the girders was then filled in with fine concrete, care being taken to see that it was packed in between the lower part of the girder and the arch, where the space was narrow.

On the top flanges of the two outside girders a bed of cement was laid to cover the rivet-heads, and upon this the brick parapets

Fig. 3.

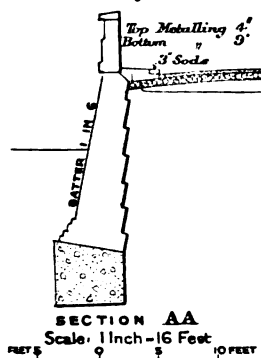
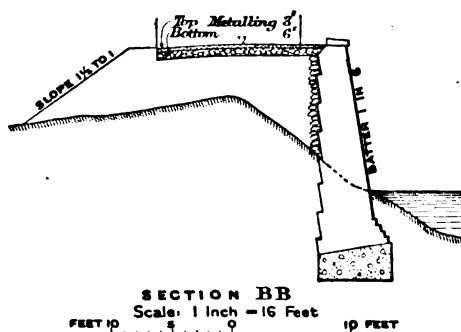


Fig. 4.



were built. They were faced on both sides with brindled brick, and were built so that when the finished road-level was obtained they stood 4 feet 6 inches high. The width at the bottom was 1 foot $8\frac{1}{4}$ inches, reducing by means of two plinth courses to 1 foot $3\frac{3}{4}$ inches. The string course was given a $2\frac{1}{4}$ -inch projection beyond the edge of the flange of the outer girder, to give a better finish. A strong blue-brick coping in cement mortar was set on the top (Figs. 2-4).

The whole of the surface carrying the roadway was asphalted with 1 inch of asphalt, put on hot in two layers, and when this was set, sods 3 inches thick were laid over it. The object of the sods was to prevent the bottom metalling of the roadway cutting into the asphalt, which would have caused leaks through the bridge; the sods in fact act as a cushion. The bottom metalling, consisting

of old bricks, was placed in position by hand and was carried right across the bridge from one parapet to the other. It thus allowed a free drainage from the centre of the bridge to the abutments, a fall being formed by the $1\frac{1}{2}$ -inch camber given to the girders, and the water-tightness being guaranteed by the asphalt. The top-metalling was 4 inches thick consisting of granite, broken to pass a $2\frac{1}{2}$ -inch ring. It was given a good finish, with a 3-inch camber, by a steam-roller.

To provide for storms, proper drainage of the road surface had to be made; on one side, by two gulleys, the pipes from which being carried through the brickwork and built in so as to discharge in the river. On the other side pipes were put on each side, under the verge, the invert being kept below the surface of the road, so that any storm-water flowed through it and discharged into the side ditches on each side.

No footpath was provided over the bridge, but the roadway was 18 feet wide with two 3-foot 6-inch verges on each side. Along the roadside a line of sods was placed, the remainder of the boundary being soiled with good earth, and sown with grass-seed.

From actual observations it was found there was no apparent deflection in the girders when the 10-ton steam-roller was going over the bridge, although there was naturally a slight vibration.

The bridge was opened to traffic on the 9th November, 1909, having thus taken, including the demolition of the old bridge, 21 weeks. It would probably have been completed earlier but for the delay of a fortnight in the delivery of the steelwork and also the dearth of bricklayers, for during most of the time the supply only equalled half the demand.

The work was carried out under Mr. W. B. Worthington, M. Inst. C.E., Engineer-in-Chief, Midland Railway Company, and Mr. John Willmot, Surveyor and Bridgemaster, Warwickshire County Council, to whom the Author, who acted as Engineer-in-Charge, is indebted for permission to submit this Paper. The general contractors were Messrs. Herbert Ashley of Mansfield, and the steelwork contractors were the Butterley Company, Ltd., Derbyshire.

(Students' Paper No. 649.).

"The Construction and Setting-Out of the Low-Level Sewer between Cannon Street and Belgrave Square."

By LAWRENCE TRENCH WILSON, Stud. Inst. C.E.

(Abridged.)

THE work consists of $5\frac{1}{2}$ miles of sewer at a depth of about 60 feet, varying in size between 7 feet 6 inches and 8 feet 6 inches internal diameter, and is part of the scheme for the enlargement of the main drainage system of the Metropolis which is being carried out under the direction of the Chief Engineer of the London County Council, Mr. Maurice Fitzmaurice, C.M.G., M. Inst. C.E., who has kindly granted permission to the Author to submit this Paper. The entire length of the section of the sewer now under review is constructed in tunnel of iron segments, by means of shields, and is lined with concrete and blue bricks. The complete low-level sewer, of which this is only a portion, extends from Hammersmith to Bow. The part of the works under consideration was let in two contracts, both being obtained by Messrs. J. Mowlem and Company, of Westminster, their tender for both contracts being £350,000. There are seven working-shafts in all, besides extra shafts to be sunk after the tunnel has been driven to form manholes, etc. The Western Section, extending from Belgrave Square to Cannon Street, will alone be dealt with in this Paper. In this section there are four working-shafts, namely, at Belgrave Square, Trafalgar Square, St. Mary's Church (Strand), and Cannon Street. All the working-shafts being in clay, with little or no water to deal with, they were sunk in timber frames.

CONSTRUCTION.

Tunnelling West of Trafalgar Square Shaft.—This drive is 4,200 feet, and, with the exception of the first 400 feet, the whole length to the middle of the Green Park, where a junction is to be made from Belgrave Square shaft, is in ballast, and is driven under com-

¹ This Paper was read and discussed at a meeting of Students in London on the 4th February, 1910.

pressed air. About 3,000 feet of this length have already been driven. Owing to the ballast in some places being free of sand with very little water to form a seal, great difficulty has been found in maintaining sufficient pressure for safe working when opening up the face for a fresh length. It must be remembered that besides the main function of driving the water away from the face, compressed air may be said to hold up the top while it is temporarily unsupported for a narrow width, when a fresh length is being got out ahead of the shield. Thus, when compressed air is employed, there is no need to pile the top, which would otherwise have to be done, and would delay the work. The top boards then are merely legged. If there is ample pressure and the ground is not loose, 4-foot top boards, 8 or 9 inches wide, may be used, one board alone being excavated for at a time. The use of 4-foot boards obviously accelerates the work, since the shield can be driven forward twice, and two rings of iron erected to every poling down of the face. Should the top be loose and treacherous, either a 2-foot top board must be used, which greatly lessens the danger of a run, or extra narrow 4-foot boards may be employed, the top being cross-poled before the 4-foot board is put up.

Each face-gang consists of seven men and a boy besides the ganger. As the tunnel advances, this gang lays a temporary 18-inch gauge road, supported on rough boarding which rests on 6-inch by 3-inch sleepers spaced in every other ring. Close to the face is a turn-out or siding for holding empty skips, iron segments on bogies, etc. When the distance to the face exceeds about 100 feet, the turn-out is shifted forwards, and a permanent road, as it is called, is laid for the electric locomotive. The rough boarding is taken up, and flat-bottomed rails, weighing 28 lbs. per yard, are spiked to the sleepers. The locomotive takes its current at 200 volts from a trolley-wire suspended from the roof of the tunnel, which is fed by a motor-generator at the top of the shaft, driven off the Power Company's mains. A motor-generator set is introduced in order to do away with a return wire. This is done by earthing the return wire, which would not be allowed with the Power Company's main. The motor-generator consists of a 53-HP. shunt-wound motor direct coupled to a 33-kilowatt compound-wound dynamo, the voltage in each case being 200. The locomotive is of 12 HP., and, with a load of eight skips, can travel about 8 miles per hour. Compressed air is supplied partly by four Brotherhood engines, and partly by an Ingersoll-Sergeant engine, all of which are driven by electricity.

East of Trafalgar Square Shaft.—For various reasons the contractors decided to drive a short length of 300 feet only in this direction as far as Charing Cross post-office in the Strand. The ground met with was blue clay, compressed air not being needed. This length has already been driven, and a junction made from St. Mary's shaft.

West of St. Mary's Shaft.—This drive down the centre of the Strand as far as Charing Cross post-office was about $\frac{1}{2}$ mile in length, entirely through blue clay, no special difficulty being met with. When the shield is in clay, no timbering is required to hold up the top and face. To accelerate progress, a groyne is first excavated down the centre of the face about 18 inches square, and hardwood piles (4 feet by 4 inches by 3 inches) wedged in front of the diaphragm plate round the cutting edge. These piles break away the clay as the shield advances. It will be seen that the bulk of the excavation is done by the piles, a very small piece only being peeled off by the cutting edge. There are $4\frac{1}{2}$ cubic yards of clay in the solid in each ring of 21 inches. This fills about 12 to 14 half-yard square skips, which are taken to the shaft by the electric locomotive. They are then picked up with scissor bales by the crane, and tipped on to the muck-stage, from which the clay is carted away to the shoot. The rate of progress in this ground was eight rings, or 14 feet, in the 24 hours, working double shifts.

East of St. Mary's Shaft.—This drive was 3,400 feet, and with the exception of a small portion at Ludgate Circus, the whole length was in blue clay. This length has already been driven and a junction has been made from Cannon Street shaft. The sewer passes to the south of St. Clement Danes Church and runs down the centre of Fleet Street. After passing Ludgate Circus, in order to avoid going too close to St. Paul's Cathedral, the sewer is deflected to the south beneath property until it emerges into Knightrider Street. Accordingly, before the sewer reached the bottom of Fleet Street, a compressed-air plant was installed at St. Mary's shaft and an iron air-lock constructed in the tunnel exactly similar to that at Trafalgar Square shaft. It was built close to the shaft in spite of the fact that nearly $\frac{1}{2}$ mile of tunnel had already been driven, and that the compressed-air plant would have to keep this enormous length charged. This was possible owing to the fact that the whole length was in clay, and to decrease the escape of air all joints down to road-level were filled up with rust caulking. Thus it will be seen that the locomotive was still able to work from the face back to within a short distance

of the shaft, which would have been impossible if the lock had been built close to the face in the usual way, in which case ponies would have to have been employed between the lock and the face. When it is remembered that a fresh pony has to be employed for every 200 feet of tunnel driven, it will be realized what an expensive item this is to the contractor.

Fleet Sewers.—At Ludgate Circus the new sewer, under compressed air, passes under the Fleet storm-relief sewer and Fleet chamber. Up to the middle of the eighteenth century a small stream, with mud banks on either side, flowed into the Thames, along what is now known as Farringdon Street and New Bridge Street. It was spanned by a small bridge, which still exists and supports part of the roadway at Ludgate Circus. In the year 1732 the old ditch was filled in and two sewers were constructed to take the drainage, the old bridge being made use of as a junction-chamber. Parallel with these sewers, but constructed at a later date, runs an 8-foot three-ring brick sewer called "the Fleet storm-relief sewer." The invert level of the storm-relief sewer was only 11 inches above the skin of the shield. In order to drive under it with safety, the inner two rings of brickwork were hacked out for a length of 9 feet on either side of the centre line of the new sewer, and ten rings of iron lining erected, the flanges of which were filled in with concrete to the former surface of the brick sewer. Since originally there was only an 11-inch clearance, and of this 8 inches had to be cut out to insert the iron lining, there only remained 3 inches clear between the under surface of the iron segments in the Fleet sewer and the correct level of the skin of the shield. In order that the lining of this sewer might be done in the dry, two stanks about 2 feet high were constructed in the sewer, one on the up-stream and the other on the down-stream side of the work, a 9-inch iron pipe being carried through the stanks to take the ordinary dry-weather flow of the sewer. The flap-valve at the mouth of the sewer seldom kept the Thames from backing up at high water, partly, no doubt, owing to small pieces of wood dropped by the men catching in the valve and preventing it from shutting. In consequence of this, and also stormy weather, the work could only be carried on for a few hours at low tide, with the result that progress was very slow. The lining of this sewer had to be done during the winter, as the sewer taps a large paved area in the north of London, which renders it dangerous in summer. The invert-level of the Fleet chamber was 4 feet 6 inches above the shield of the new sewer, and this was also subject to the rise and fall of the river, there being about 12 feet of water at high tide. It will be

realized that this was a most treacherous piece of work, as there was the double danger that either the invert might collapse owing to the removal of the foundations, or else that it might be blown up by the compressed air. Various suggestions were put forward to lessen these dangers, such as strutting down the invert with timber, or laying a blanket of clay all over the invert. In the end, however, nothing was done in the chamber itself, but very great care was taken in excavating the ground ahead of the shield. Only 6 inches forward was taken out at a time; the face was then timbered up and the shield driven forward. The work was carried out successfully in this manner, and it took about 4 days to pass under the chamber. Curiously, the foundations of the old bridge were not found; the only brickwork that had to be cut away was the bottom of the storm-relief sewer. It was not until the shield had almost passed under the Fleet chamber that the bed of the old stream was found. This probably indicates that the bridge was built in the dry and the stream afterwards diverted to pass through it.

Lining.—As previously stated, junctions have been made both east and west of St. Mary's shaft. Accordingly, the length from Trafalgar Square to Cannon Street is now in course of being lined with concrete and blue bricks. The invert of the sewer is lined with one ring of blue bricks, and the space between the flanges behind the brickwork is filled in with concrete. The arch is constructed of 3-to-1 granite concrete.

SETTING-OUT.

The longest distance between any two shafts on the works is 7,000 feet, namely between Belgrave Square and Trafalgar Square. It is obvious that accurate surveys are required in order that the tunnel may join correctly for both line and level.

Surface Works.—There are two methods of surveying the line of sewer, namely, either surveying the actual line of sewer or traversing about the line. By the first method the actual intersection points of the sewer, having been scaled off the plan by two or three tie measurements, are marked on the ground by 7-inch iron spikes driven down flush with the road surface. The theodolite is set up over punch-marks in these spikes, and the angles and distances are measured between them. This method is by far the simplest, since no calculations are required after the survey has been made; but it suffers from the great disadvantage that the road may be taken up for some purpose and the point lost. Further, it is in-

applicable where the sewer passes under property, as in this case the intersection-points are frequently inaccessible on the surface. By using the second method, the actual intersection-points need never be marked on the ground at all, but the angles and distances are calculated from a traverse which is made to follow the line of sewer as closely as possible. Traverse points are fixed so that there shall be as few angles as possible, provided also that there shall be no very short lines. They consist of punch-marks on iron disks, about 2 inches in diameter and $\frac{1}{2}$ inch deep, let into the granite curb-stone. The curb-stone is the part of the road least likely to be moved, and is chosen in preference to the pavement, which is liable to be taken up by telephone or electric-light companies. The latter system is more satisfactory than the former, and, whether the sewer passes under property or not, it is advisable to use it, in spite of the mass of calculations which it involves.

Measurements of Angles.—The theodolite used has a 7-inch circle fitted with two micrometers, and has no vertical circle. The micrometer is divided into ten-second divisions which can be estimated farther by the eye to one second. It is also fitted with sliding plates for bringing the centre of the instrument over the punch-mark in the spike or disk. Attached to the sliding plates are three pointed legs for setting up on wooden byats in the tunnel, and on the top of the parallel plates is a small sharp-pointed cone about $\frac{3}{8}$ inch high which is the true centre of the instrument, and is used in the tunnels to bring the theodolite exactly under the plumb-bob by means of the sliding plates.

Measurement of Distances.—For this purpose a 100-foot steel band is used. Prior to the commencement of such a survey some standard is agreed to between the engineers and contractors. The British standard length of 100 feet at Trafalgar Square is measured at 62° F., which was a convenient standard temperature for use on these works, as the temperature in the tunnel throughout the winter and summer happens to remain very nearly constant at 62° F. This standard temperature was therefore adopted, and it was decided that the band should be exactly correct at 62° F. and with a pull of 10 lbs. The pull is applied by means of a spring-balance, which is accordingly used throughout the work for measuring all distances. All lines measured at other temperatures had to be corrected to the "true length," i.e., what they would have measured at 62° F. The formula used for this correction is:— $\frac{1}{16}$ inch per 10° per 100 feet. The amount given by this formula has to be subtracted from measured lengths if the temperature is less than 62° and added if the temperature is more than 62°.

Level Correction.—All lines must also be reduced to their true horizontal measurements. This is effected by taking a level generally at every 100 feet and also where sudden variations occur in the road-surface. The formula used for this correction is:—

$\frac{\text{Rise}^2}{\text{twice measured length}}$ which has always to be subtracted from the measured length. The temperature and level corrections may seem small, but on long lines the total correction will often be a very appreciable amount. As an example, the line down Constitution Hill may be cited. This line is about 1,850 feet long, and as the temperature was 28° F. on the night it was measured and the slope is considerable, the total minus correction amounted to 8 inches; of this the temperature correction was 5 inches and the level correction 3 inches. It may seem needless to work with such accuracy throughout the surveys, but when it is borne in mind that the distance between some of the shafts is nearly $1\frac{1}{2}$ mile and that the base-line in the shaft for transferring the survey below is only 10 or 12 feet, it will be evident that too great care cannot be taken. Working under these conditions, the engineers and contractors are able to agree the lengths of lines to $\frac{1}{8}$ inch or $\frac{1}{4}$ inch, although measured quite independently on different nights and at different temperatures. The angles are also agreed to within 2 or 3 seconds. It might be mentioned that the whole of the $5\frac{1}{2}$ miles of surface survey had to be done at night, owing to the heavy traffic making work by day impossible, and that part of it in the Strand and Fleet Street could only be done on Saturday nights owing to the incessant stream of newspaper traffic throughout every other night of the week.

Transferring the Survey below.—The theodolite is set up on the centre-line of the sewer prolonged across the shaft, and two wires, which can be moved in either direction by means of a fine tangent screw, are set on line. These wires should be as far apart in the shaft as possible, as the accurate setting out of this base depends largely upon a good junction. Attached to the wires at the bottom of the shaft are 50-lb. plumb-bobs hanging in buckets of water—not in oil, as is frequently recommended in text-books. The theodolite should be set up about 15 feet from the nearest wire, so that each wire may be in a completely different focal plane. The instrument as stated above should, if possible, be set up on the further side of the shaft from the intersection-point or disk, thus rendering it unnecessary to transit the telescope when putting the wires in line. This avoids the necessity of correcting for collimation and the errors in horizontal axis, which would otherwise have to be taken into account. After the wires have been put into line

and the necessary measurements taken to the wires for connecting the bottom survey with the top, the theodolite is set up on a byat fixed across the tunnel at about the same distance from the wires as on the surface. It is then shifted about by means of the sliding plates until it is exactly in line with the wires. In this case it is, as a rule, impossible for the instrument to be set up so as to avoid transitting, owing to the sewer being on a curve through the shaft, and accordingly the errors have to be allowed for in the usual way.

Temporary Work in the Tunnel.—These are the lines on which the shield is actually driven. They are always just ahead of the permanent lines, and are kept as close as possible to the true line. They consist of plumb-lines hanging in file marks on $\frac{1}{4}$ -inch iron dogs, which are bolted to the tunnel segments, and are given to the ganger at about every thirty rings, to enable him to keep the shield on line. As the shield advances and fresh lines are put up, the back lines drop out of use.

Permanent Lines in the Tunnel.—These should be an exact reproduction of the calculated lines and angles from the surface survey. Usually they are put in at angle stations only; occasionally, however, on long straights, intermediate points are established for convenience in sighting. Permanent points in the tunnel are best fixed by embedding iron plates about 3 inches or 4 inches square in blocks of concrete built in the invert of the tunnel. A small centre punch-mark on the iron plate denotes the exact position of the intersection-point. The real function of these points is to correct the temporary lines, in order to ensure a correct junction. The temporary lines are put in under circumstances of great difficulty and haste, and cannot, therefore, be relied upon to be correct for long lengths of straight or curve. Accordingly, as soon as the shield has passed an intersection-point, the permanent lines are carried on up to that point, and the temporary lines are corrected. It may be of interest to mention that the error of the lines in the junction between Trafalgar Square and St. Mary's shaft, a distance of 3,000 feet, was $\frac{7}{8}$ inch; and between St. Mary's Church and Cannon Street shafts, a distance of 6,000 feet, the error was 1 inch.

In conclusion, it may be mentioned that the work is being carried out for the Contractors by Mr. Rowell, and is under the supervision of Mr. J. E. Worth, M. Inst. C.E., District Engineer, London County Council, Mr. C. F. Clough, M. Inst. C.E., acting as Resident Engineer.

OBITUARY.

SIR JOHN AIRD, Bart.,¹ senior member of the well-known contracting firm of John Aird and Company, died at his residence, Wilton Park, Beaconsfield, Bucks, on the 6th January, 1911, aged 77. The contracting business of John Aird and Sons, the former name of the firm, covers the whole of the latter half of the last century. John Aird was associated with his father in 1851, on reaching his eighteenth year; he had spent the greater part of his boyhood at Greenwich, and was educated there and at Southgate. As regards his apprenticeship in engineering, he could not have had a better schooling than the one he received in association with his father's business. At this period, his father commenced carrying out important undertakings, among which may be mentioned work in connection with the Great Exhibition in Hyde Park. At about the same period Messrs. Aird extended several gasworks in this country, and were the contractors for new works of the same class both in London and in the provinces. Their sphere of activity was not, however, limited to Great Britain, but extended to the Continent of Europe and to distant countries; their name is associated with the early installation of waterworks and gasworks in Amsterdam, Berlin, and Copenhagen, and they also put down gasworks in France, Italy, and Brazil. Amongst other large works of the same nature carried out by them in connection with various contracting firms, were the Calcutta Waterworks, the Millwall Docks, and a large amount of civil engineering work in Sardinia.

John Aird, after the death of his father in 1876, entered into partnership with Messrs. Lucas Brothers, the new firm being styled Lucas and Aird. The firm built various extensions of the District Railway, the Metropolitan Railway, and the St. John's Wood Railway; the Royal Albert Docks, the Tilbury Dock, the East and West

¹ This memoir has been reprinted, with certain alterations, from *Engineering*, 13 January, 1911, by permission of the Editors.

India Dock Extension, the Hull and Barnsley Railway and Dock, the West Highland Railway, besides many other new railway lines, railway-widening works, docks, water, sewage, and drainage works and railways in the provinces and in foreign countries.

The crowning work accomplished by Sir John Aird as a contractor was the construction of the Assuan Dam on the Nile, the Engineer for which was the late Sir Benjamin Baker, Past-President. The work is fully described in the Paper by Mr. M. Fitzmaurice, C.M.G., published in the Proceedings.¹ Preliminary operations were commenced at the site of the dam in April, 1898, and in the early summer of 1899 the number of men employed on the undertaking reached a total of 13,000. The dam has a length of about 2,200 yards; it has 180 sluice-gates, and its maximum height from the foundation is about 130 feet. The total amount of granite masonry its construction involved was about 1 million tons. The effect of the dam is to convert the River Nile above Assuan into an immense reservoir, which, when full, contains about 38,000 million cubic feet of water. Its construction took about 4 years only. The Assiut Barrage was carried out at the same time.

Among the later undertakings carried out by Sir John Aird and Company in this country may be mentioned the Royal Edward Dock at Avonmouth, which was commenced in March, 1902. The works at Avonmouth increase the deep-water area of the port by 30 acres, and reclaim from the sea a large tract of land through the tipping of the excavated material on the mud slopes of the Severn, thus raising them much above the level of the sea. The firm is occupied in Egypt in raising the Assuan Dam, and have lately completed a barrage at Esneh and the Menufieh Regulator.

Sir John Aird was also well known as a connoisseur of art, and owned an admirable collection of valuable paintings, water-colours, and statuary. He was created a baronet in 1901. He was a Member of Parliament for North Paddington from 1887 to 1905. In 1900 he was elected the first Mayor for the newly-formed Borough of Paddington. He was, moreover, Commissioner of Lieutenancy of the City of London and Lieut.-Colonel of the Engineer and Railway Volunteer Staff Corps. He was also a member of the Iron and Steel Institute. Sir John is succeeded by his elder son, John Aird, Assoc. Inst. C.E., who was born in 1861.

Sir John Aird was elected an Associate of The Institution on the 6th December, 1859, and at one time served as an Associate of the Council.

¹ Minutes of Proceedings Inst. C.E., vol. clii, p. 71.

WALTER BEESLEY, born at Dulwich on the 28th November, 1864, died on the 8th November, 1910. Trained under his father, the late Mr. Frederick Beesley, in the sanitary branch of the profession, he was subsequently engaged, with his father and later as senior partner in the firm of Beesley, Son and Nichols, in the design and construction of water-supply and sewerage systems for many towns in England, Brazil, and South Africa, the works including Chertsey, Leatherhead, Newmarket, Worcester, Rio de Janeiro, Manaus, Campos, Lourenço Marques, and Kalk Bay. Mr. Beesley was elected an Associate Member on the 9th January, 1894, and was transferred to the class of Members on the 16th December, 1902.

BAGOT WILLIAM BLOOD, born at Leamington on the 6th November, 1844, died at Dalkey, Co. Dublin, on the 4th February, 1910. He joined the Indian Public Works Department in 1866, and subsequently became Superintendent of Way and Works of the Rajputana-Malwa railway, afterwards absorbed in the Bombay, Baroda and Central Indian system. For some years before his retirement in 1893, he held executive engineering charge of several hundred miles of main line and branches on the latter system. He was for some years Colonel in command of the Rajputana-Malwa Volunteer Rifles. He was elected a Member of The Institution on the 5th March, 1878.

THOMAS WILLIAM ROBERT HUGHES, born on the 5th June, 1853, died on the 13th October, 1910. He was trained under Messrs. J. and W. Dudgeon, marine engineers and shipbuilders, and served 6 years as Engineer in the Peruvian Navy. In 1881 he became Engineer of the First class in the Argentine Navy, and was subsequently made Chief Inspecting Engineer, and later Technical Adviser to the Argentine Legation, in which capacity he was responsible for all material for the Public Works Department, as well as construction work and machinery for the Navy. Mr. Hughes was elected a Member of The Institution on the 6th March, 1906.

AUGUSTUS HAMILTON JACOB, M.A., M.A.I., born on the 14th June, 1840, in Dublin, died at Bournemouth on the 25th June, 1910. Graduating with honours in Trinity College, he obtained his early practical experience on railway and municipal work at home, and was appointed to the Public Works Department of Travancore in 1870. He served 10 years as Assistant Engineer and over 15 years as Chief Engineer, in which latter capacity he carried out a number of important public works, and designed and initiated the Kadayar irrigation scheme, the construction of which was well advanced when he retired in 1896. Mr. Jacob was elected a Member of The Institution on the 1st March, 1881.

JOHN KYLE, born in Scotland in 1861, died on 9th January, 1911, at Colombo, Ceylon, where he occupied the position of Resident Engineer of the Colombo Harbour Works. Mr. Kyle obtained his practical training as pupil and later as assistant, with Messrs. Bell and Miller, of Glasgow. He was appointed in 1884 assistant engineer, Colombo Harbour Works, his father¹ at that time occupying the position of Resident Engineer. Subsequently Mr. Kyle became Chief Assistant Engineer of the Harbour Extension Works under Mr. J. H. Bostock, Resident Engineer, and on Mr. Bostock's decease, Mr. Kyle succeeded him. He was much esteemed in Colombo for his quiet demeanour and high character.

Mr. Kyle was elected an Associate Member of The Institution on the 1st March, 1887, and was transferred to the class of Members on the 6th November, 1894.

JAMES MELDRUM, born at Colinsburgh, Fifeshire, on the 19th September, 1855, died at Colombe Lodge, Ealing, on the 15th December, 1910. Educated at St. Andrews and Glasgow Universities, he was trained under Messrs. Bell and Miller, of Glasgow, and subsequently was responsible for the construction of important railway works abroad, including the Condé d'Eu line in Brazil, the Manila Railway, the first in the Philippines, and the Buenos Aires Great Southern Railway. For 12 years he was

¹ Minutes of Proceedings Inst. C.E., vol. clxiv, p. 408.

manager of the Foreign department of Messrs. S. Pearson and Sons, contractors, supervising large railway, harbour and dock contracts in China, Mexico and South America, and latterly he had an extensive practice as a Consulting Engineer and served on the directorate of several companies.

Mr. Meldrum was elected an Associate Member of The Institution on the 4th May, 1886, and was transferred to the class of Members on the 7th December, 1897.

JASON RIGBY was born in Dublin on the 24th July, 1846. He entered Trinity College in 1864, and in due course graduated in Arts and obtained the licence in Engineering. In 1868-9 he was a pupil of Mr. James Price, M. Inst. C.E., and was afterwards employed by him in the general work of the Midland Great Western Railway of Ireland. In 1871-2 he was assistant to Mr. Tait, County Surveyor of Antrim, and had sole charge of a district.

In 1872 he sailed for Brazil, and was employed as first assistant on the Parana and Matto Grosso railway surveys under the late Captain C. Palm and afterwards under the late Mr. William Lloyd. Whilst engaged on this work he penetrated with his companions into parts of Brazil previously unexplored, and, with possibly one exception, is known to have been the last survivor of this expedition. In 1875 he married Miss Anne Alice Andrews of Hampstead.

In 1876 he returned to Brazil on behalf of Sir Charles Fox and Sons to survey the construction of the Imperial Brazilian Natal and Nova Cruz line in Rio Grande do Norte, and from September, 1878, he had charge of the construction of that railway until his appointment in 1883 as Resident Engineer for Messrs. Sir Douglas Fox and Partners. As soon as the line was completed the Board invited him to become General Manager and Chief Engineer, a position he held until 1885.

The late Mr. Frank Parish, who was Chairman of the Great Western of Brazil Railway Company, then offered him the Managership of that line, an appointment which he held until 1891, residing in Pernambuco, where yellow fever was very prevalent in those days. He left the Great Western of Brazil Railway to take charge of the Minas and Rio Railway, and shortly afterwards he undertook the management of the Entre Rios Railways, living at Parana. In 1894, again at the invitation of Mr. Frank Parish, he became General Manager of the Buenos Ayres and Rosario Railway, since

merged into the Central Argentine Railway. He held this position until 1899, when he came home to accept a seat on the Board of that Company. In the same year he became a Director of the Buenos Ayres Great Southern Railway Company, and on the death of Mr. Frank Parish in 1906, he was elected Chairman of that Company.

At the time of his death Mr. Rigby was a Director of a number of railway companies in South America. He died suddenly at his residence, Holmleigh, Putney Hill, on the 9th September, 1910, aged 64.

Mr. Rigby was elected an Associate of The Institution on the 1st February, 1876, was subsequently placed in the class of Associate Members, and was transferred to the class of Members on the 13th March, 1883.

JAMES STELFOX, born on the 23rd June, 1842, died at Belfast on the 11th September, 1910. Trained under his father, the late Mr. James Stelfox, in the Belfast gasworks, he later succeeded him as Chief Engineer, a position which he filled until within a few years of his death. During his tenure of office the gas undertaking developed considerably, and amongst other improvements carburetted water-gas and prepayment gas-meters were introduced. He was elected a Member of The Institution on the 1st December, 1896.

GEORGE CARELESS TREWBY, formerly Engineer-in-Chief and Consulting Engineer to the Gas Light and Coke Company, died suddenly from heart failure at his residence, Fenton House, Hampstead, on the 19th July, 1910, aged 71. Entering the Gas Light and Coke Company's works as a pupil in 1853, he subsequently went out to Constantinople as engineer in charge of gasworks for the Ottoman Government, returning to England 2 years later to take charge of the Brentford gasworks. Early in 1865 he rejoined the staff of the Gas Light and Coke Company, and acted as Resident Engineer at the Horseferry Road station until 1870. He was then transferred to the Beckton works, for the construction and organization of which he was largely responsible. He was in charge there during the memorable strike of stokers in 1872. For 7 years he supervised the carbonization processes at the

several manufacturing stations, and for the last 20 years of his service he was the Company's Engineer-in-Chief. He was responsible for all extensions and reconstructions carried out during his tenure of office, and introduced a number of labour-saving appliances and other improvements at the various works under his charge. The coal-pier at Beckton and its mechanical equipment were designed by him, and are described in the Proceedings.¹ Mr. Trewby gave important evidence before several Parliamentary and Departmental Committees on matters connected with the gas industry. He retired from the service of the Company in 1904.

Mr. Trewby was a member of the earlier gas societies, and was a founder and the first President of the Institution of Gas Engineers. He did good work in connection with various charitable bodies, with the direction of which he was associated.

He was elected an Associate of The Institution on the 6th March, 1866, and was transferred to the class of Members on the 27th April, 1875.

HENRY HAY WAKE, Engineer to the River Wear Commissioners, died in Sunderland, his birthplace, on the 17th February, 1911, aged 66. A pupil of the late Mr. Thomas Meik, then Engineer to the Commissioners, he succeeded that gentleman as engineer-in-charge in 1868. During his tenure of that office, he carried out extensive improvements of the port of Sunderland, including the removal of rock obstructions at the entrance, the reconstruction of docks, erection of warehouses and other structures, and the design and construction of two large piers or breakwaters, one of which is not yet completed. In conjunction with the late Mr. P. W. Meik, he also designed the new piers at Seaham Harbour.

Mr. Wake was elected an Associate of The Institution on the 24th May, 1870, and was transferred to the class of Members on the 10th March, 1874.

¹ Minutes of Proceedings Inst. C.E., vol. lxi, p. 318.

JOHN EDWARD WOLFE, born on the 19th April, 1850, died on the 15th December, 1910. Trained under the late Mr. Wm. Bouch he was later assistant to Messrs. T. and C. Hawksley, and became successively Manager and Engineer of the Alagoas Railway, Brazil, Engineer of the Sunderland and South Shields Water Company, and General Manager of the United Railways of Havana. His last engagement was to take charge, in 1906, of the São Paulo and Rio Grande and other lines for the Brazil Railway Company. Mr. Wolfe was elected a Member of The Institution on the 4th March, 1884.

JOHN BEVERIDGE, born on the 11th July, 1856, at Auckland, New Zealand, died at Sydney on the 8th May, 1908. After serving as a cadet in the Public Works Department, he practised for some time as a licensed Surveyor and was subsequently appointed assistant City Engineer of Auckland. In 1890 he joined the staff of the Sydney Board of Water Supply and Sewerage, and at the time of his death held the position of District Engineer for Sewerage Construction. He was elected an Associate Member of The Institution on the 4th December, 1888.

THOMAS JOHNSON BRITTEN, born on the 6th March, 1858, in Herefordshire, died at Johannesburg on the 24th October, 1910. He was apprenticed to Messrs. John Elder and Company, of Glasgow, and subsequently was employed by Messrs. Hornsby and Sons, of Grantham, as their representative in South Africa. Subsequently, he practised in Johannesburg on his own account, and was General Manager of the Wolluter Gold Mines, and Consulting Engineer to Messrs. Abe Bailey and Company. He was elected an Associate Member of The Institution on the 1st December, 1896.

JAMES BULL, born at Bedford on the 13th August, 1844, died at Hove on the 14th January, 1911. The greater part of his career was passed in Spain, where he was engaged in mining and railway work, both for contractors and as Engineer and Manager of the

Buitron Company's mines and railways, and other undertakings. He was an Alderman and a Justice of the Peace at Hove.

Mr. Bull was elected an Associate Member of The Institution on the 6th December, 1881.

HERBERT NICHOLAS FUNNELL, born on the 22nd January, 1880, was killed in a tramcar accident at Seneca Falls, New York State, on the 5th June, 1910. He served a three years' apprenticeship with Messrs. James Simpson and Company, and after some years' varied experience in this country he went to the United States in 1907, where he served short engagements with the New York Edison Company, the Sheridan Iron Works, the Cleveland Automatic Machine Company, and the Goulds Manufacturing Company, with whom he was employed when he met his death. Mr. Funnell was elected an Associate Member of The Institution on the 5th December, 1905.

THOMAS MUSGRAVE HEAPHY, born in London in 1843, died at St. Leonards-on-Sea on the 29th March, 1910. Trained on the staff of the late Mr. William Wilson, he later engaged in consulting practice. In 1874 he was appointed scientific adviser, engineer and inspector to the Phoenix Fire Office, and was the author of the Phoenix rules for electric-light and power installations. As a consulting engineer, he designed a large number of electrical installations. He retired in 1904. Mr. Heaphy was elected an Associate Member of The Institution on the 6th February, 1893.

BENJAMIN LANGFORD FORSTER POTTS, born at Blackheath on the 29th July, 1839, died on the 8th September, 1910. On completing his training, he became an assistant to the late Mr. E. A. Cowper,¹ with whom he remained until 1876. After a few years' service at the Patent Office, he practised as a Consulting Engineer, retiring in 1904. He was elected an Associate Member of The Institution on the 4th April, 1876.

¹ Minutes of Proceedings Inst. C.E., vol. cxiv, p. 182

PATRICK JOSEPH PRENDERGAST, born at Clifden, County Galway, in March, 1853, died at Galway on the 1st February, 1911. Graduating in engineering at Queen's College, Galway, he was occupied in early years on constructional work for the Great Northern and Midland Great Western Railway Companies of Ireland. In 1892 he became Borough Surveyor of Athlone, and in 1902 he was appointed County Surveyor to the West Riding of Galway, resigning in October, 1910.

Mr. Prendergast was elected an Associate Member of The Institution on the 5th February, 1895.

FRANCIS BERTRAM ROBINSON, born at Skipton-in-Craven, Yorkshire, on the 13th March, 1880, died at Pará, Brazil, on the 28th September, 1910. Educated at Charterhouse, and at the Yorkshire College, Leeds, he served as Assistant Engineer on the Midland Railway staff at Heysham Harbour from 1906 to 1909, when he went out to Brazil as Engineer on the staff of the Pará Construction Company. Mr. Robinson was elected an Associate Member of The Institution on the 6th February, 1906.

ROBERT SHARPE, born on the 9th November, 1848, died on the 8th May, 1910, at Belfast, where he was Engineer and Manager of the Corporation Gasworks. Apprenticed to Messrs. Laidlaw and Sons, of Glasgow, gas and water engineers and contractors, he subsequently assisted the late Mr. William Foulis on gasworks at Glasgow and in Spain. He was appointed Assistant Engineer at Belfast, in 1879, and succeeded Mr. James Stelfox as Engineer in 1907. Mr. Sharpe was elected an Associate Member of The Institution on the 5th April, 1887.

CHARLES STANLEY WHITE, born at Bradford, Yorkshire, died at his residence, Holme Chase, Heathfield, Sussex, on the 1st January, 1911. Trained under Sir Douglas Fox, Past-President, as pupil and assistant, he was occupied on railway work at Wigan and the Metropolitan extension of the Great Central Railway. Trans-

ferring his services to the contracting firm of Sir Thomas Firbank, he was subsequently engaged on railway contracts at Manchester, Basingstoke, Cirencester, Park Royal, Somerton, and other places.

Mr. White was elected an Associate Member of The Institution on the 5th April, 1892.

JOHN HOWARD COLLS, born on the 30th August, 1846, died at Buenos Aires on the 29th December, 1910. Mr. Colls was joint Chairman and Managing Director of George Trollope and Sons and Colls and Sons, of which latter firm his father was the founder. He had considerable experience as a builder and contractor, and was a member and office-bearer of the Institute of Builders, the London Master Builders' Association and other bodies.

He was elected an Associate of The Institution on the 10th January, 1905.

CHARLES FREDERICK HART, born at Brigg, Lincolnshire, on the 26th October, 1828, died at Devizes on the 11th September, 1910. He obtained his training on the Manchester, Sheffield and Lincolnshire railway, and was subsequently engaged on the construction of the Devizes branch of the Great Western railway. For 22 years he acted as Secretary of the Berks and Hants Extension railway, now part of the Great Western railway system, and for 17 years until 1899, when he retired, he was Engineer to the Kennet and Avon Canal. Mr. Hart was elected an Associate of The Institution on the 6th April, 1875.

CAPTAIN THOMAS BRIDGES HEATHORN, R.A., died at 10, Wilton Place, S.W., on the 23rd January, 1911, aged 80. Trained at Putney College for the engineering profession, he afterwards entered the East India Company's military school at Addiscombe. He obtained his lieutenant's commission in 1850, and served throughout the Crimean War in the Turkish Contingent, holding the position of Deputy Quartermaster-General of Artillery. He also served in the Indian Mutiny campaign, was present at Kotah and other actions, and was mentioned in despatches. In 1868 he served on the Armaments Committee of the War Office,

and in later life, he became a director and Chairman of the South Metropolitan Gas Company, and was the inventor of several patented appliances in connection with ordnance machinery.

He was elected an Associate of The Institution on the 6th December, 1881.

* * The following deaths have also been made known between the 11th February and the 1st June, 1911 :—

Honorary Member.

CHANUTE, OCTAVE ; *died* 23 November, 1911.

Members.

AIREDALE, Lord ; <i>died</i> 16 March, 1911.	HALLETT, JOHN HARRY ; <i>died</i> 10 April, 1911.
BELL, HENRY PURDON ; date of death not known.	MASSIE, JAMES ; <i>died</i> 8 March, 1911.
BRIGGS, ROSWELL EMMONS ; <i>died</i> May, 1911.	NEATE, CHARLES ; <i>died</i> 29 May, 1911.
COKE, GEORGE ELSLEY ; <i>died</i> 14 May, 1911.	SCHWITZER, JOHN EDWARD ; <i>died</i> 23 January, 1911.
CROUDACE, CHARLES HUTTON ; <i>died</i> 20 May, 1911.	SWAN, ARCHIBALD ALEXANDER ; <i>died</i> 12 February, 1911.
DAWSON, WILLIAM ALFRED ; <i>died</i> 24 February, 1911.	UNWIN, HOWARD ; <i>died</i> 26 February, 1911.
GLASS, JAMES GEORGE HENRY ; <i>died</i> 21 April, 1911.	VAUGHAN, CEDRIC ; <i>died</i> 18 February, 1911.

Associate Members.

DAWSON, JAMES HENRY ; <i>died</i> 19 February, 1911.	MACGREGOR, ALEXANDER ; <i>died</i> 18 February, 1911.
EUNSON, JOHN ; <i>died</i> 4 April, 1911.	O'CONNELL, JAMES CHARLES ; <i>died</i> 15 March, 1911.
GREENHILL, THOMAS ARTHUR ; <i>died</i> 20 April, 1911.	RISTORI, EMANUEL JOSEPH ; <i>died</i> 21 May, 1911.
GROVE, EDMUND ; <i>died</i> 11 February, 1911.	SEALY-ALLIN, LIONEL ; <i>died</i> 12 April, 1911.
LAUTOUR, EDGAR DE ; <i>died</i> 11 May, 1911.	THOMAS, CHARLES HENRY ; <i>died</i> 29 April, 1911.

Associate.

BIRT, Sir WILLIAM ; *died* 18 May, 1911.

SECT. III.

ABSTRACTS OF PAPERS IN SCIENTIFIC TRANSACTIONS
AND PERIODICALS.*New Method of Consolidating Macadamized Roads.* C. GUILLET.

(Le Génie Civil, Paris, 1910, vol. lviii, pp. 182-5.)

It has long been known that the ordinary steam-roller, when used for the consolidation of road-metalling, causes extensive breakage, but, notwithstanding this, the surface produced lasts well, so long as the pressure to be sustained is vertical. Now, however, a remedy has to be sought for the rapid deterioration caused by the tangential pressure brought into play by the driving-wheels of the motor-car, and it is suggested that this may lay in reducing such breakage and thus making it possible for each individual stone to come more directly into contact with its neighbours in the same plane, and so derive better lateral support from them. A novel machine, depending for its action upon lifting and again dropping a series of rammers on the surface to be treated, is described. In the actual machine which has been tried each rammer or "stamp" weighs 356 lbs., and covers a surface of 5.4 square feet, while the height of drop is 7.9 inches. Four experimental lengths, each of about 550 lineal yards, have been consolidated by this machine near Mélnun, and adjacent to these, similar patches, rolled in the ordinary manner, have been laid down for purposes of comparison. The work was only completed in the autumn of 1910, and thus insufficient time has elapsed to afford a test under everyday conditions, but while 90 per cent. of the metal before use was too large to pass through a 1.18-inch diameter ring, a sample taken from the rammed surface contained 60 per cent. of this size as against 45 per cent. for one taken from the rolled length, thus showing that the breaking action of the rammer was considerably less than that of the roller. Again, with the rammer, 20 per cent. of binding material was required, whereas only half this quantity was needed with the roller, this fact tending to show that the surplus required was obtained from pulverizing the metal itself. The Author thinks that with heavier rams and a coarser-broken metal a still more durable surface might be expected.

I. C. B.

*Comparison between Macadam and Overburnt Blue-Brick Road-Metalling.*¹ D. A. VAN HEYST.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 873-5.)

The Author made a number of studies of Dutch roads under his control with a view to establish which type of metalling was the more favourable both as regards initial expense and cost of upkeep. The results of his observations are recorded in tabular form. He concludes that, although the macadam type of metalling is at present preferred, there is much to be said in favour of brick roads. They are practically free from dust in summer and from mud in winter. Their cost of upkeep and repair, including wages, is not very much more, being £42 per mile per year against £33 for macadam metalling. The Author is of opinion that overburnt blue bricks form the better type of road-metalling, but that the means at the disposal of the authorities finally determine the type used.

H. C. A. T.

Reinforced-Concrete Railway-Bridges. J. M. ZAFRA.

(Revista de Obras Publicas, Madrid, 1910, pp. 533-5.)

For narrow-gauge railways economy in cost is indispensable, and the use of reinforced concrete, compared with masonry, and still more with steel, enables cost to be reduced in the majority of cases. The Author examines the objections to the use of this material on account of the dynamic effects which passing trains may produce in course of time.

The rapid transition from a state of rest under the simple action of dead load to the dynamic state provokes a series of vibrations, the number depending on the length and rigidity of the work, and the range on the abruptness of the change, i.e., mass and velocity of the train relative to mass and rigidity of the work. The motion of the locomotive and the rail-joints produce blows, and each provokes other series of vibrations which, synchronizing, may have a wide range. In proportion as rigidity increases the number of vibrations per unit of time increases and their range diminishes. The more rigid the structure the smaller the range and less the total number of vibrations. Reinforced-concrete work under equally admissible conditions of span and load presents far greater mass and rigidity than metallic structures. Diagrams of several tests are given. Small vibrations can only be explained by an influence more powerful than mass and rigidity, viz., solidity. The solidity of reinforced concrete is complete, the whole participates more or less, always to some extent, in the effect of a local cause, and all the

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxxi, p. 490.

elements in greater or less degree contribute to counteract and absorb that effect. If dynamic vibrations are very much less in reinforced concrete than in metal, their effects over short or long periods are much less to be feared. Evidence can be adduced to show that constant exposure to shocks does not diminish or cause the loss of contact between the concrete and the reinforcement.

The adhesion in reinforced-concrete sleepers after several years' heavy traffic has been measured by a French Commission with results far superior to the 300 to 500 lbs. per square inch estimated for 3-months-old concrete, from which it may be inferred that the primitive adhesion has increased with time, although under a constant state of trepidation incomparably more violent than that to which a structure entirely of reinforced concrete can be subjected. Lastly, the reinforced-concrete works, 15 years old, on the line at the entrance to the Simplon Tunnel, where the traffic is heavy, have been found, after careful examination, in perfect condition. They are in general of small span, in which the change from a static to a dynamic state exercises greater influence. Except perhaps in the case of the chemical conservation of concrete subjected to the influence of sea-water, a well-made concrete is unalterable and maintains unaltered the armament it surrounds: this is proved by the clean state of the metal when a piece is broken up.

C. A. F.

Reinforced-Concrete Railway-Bridge at Malaga. J. M. ZAFRA.

(Revista de Obras Publicas, Madrid, 1910, pp. 536-8.)

The span of reinforced-concrete girder railway-bridges did not exceed 70 feet until the construction at Malaga, in Spain, of the bridge over the Velez, more than 85 feet long, designed by the Author. The river is crossed by five spans of 32 feet 9 inches and two of 86 feet 7 inches, the latter being best described as metallic structures embedded in concrete. The girders are 7 feet 3 inches deep, $\frac{1}{2}$ of the span, and 5 feet apart; the top and bottom flanges are formed of two, three, and four flat bars, on edge, 8 inches by $\frac{3}{8}$ inch, wrapped with No. 6 S.W.G. wire 10 inches pitch. The verticals divide the girder into twelve bays, and consist of one and, in cases, two rolled joists 4 inches deep. From the centre of the bottom flange the diagonals cross each bay to the top of the next vertical, and are formed of two pairs of 1-inch to $1\frac{3}{8}$ -inch square rods, single or double, their ends bent round the bolts which pass through the flanges. This skeleton frame, considered apart, works under an apparent stress of dead load, moving load, and the vertical component of the wind-pressure (35 lbs. per square foot) of 7.2 tons per square inch in the flanges, at less than 5.7 tons in the diagonals, 4 tons in the bolts, and 3.8 tons in the rivets at the joints: in the verticals the strain is 430 lbs. per square inch on the concrete only. The whole frame

is buried in concrete, the dimensions of which are 12 inches by 12 inches in the diagonals; the verticals are 12·5 inches by 12 inches and 24 inches by 12 inches. The floor, 13 feet wide, joining the top flanges and overhanging them 4 feet on each side to form the footways, is 4 inches thick in the centre between the girders and at the extremities, from which three points it increases in thickness to 9 inches, where it joins the concrete surrounding the top flanges. The obtuse angle at the junction is still further filled in by a gusset of concrete, making a total thickness there of 16 inches. The effect of the weight of concrete in the floor is to relieve considerably the stress in the upper flange and to raise the centre of gravity of the whole work, although these variations are at the expense of the bottom flange, where the apparent stresses will remain about the same as when the frame was considered alone. The chief gains are suppression of secondary stresses, sagging, transverse deflections, etc.; above all, consolidation of the whole. The metre-gauge rails are carried on sleepers fastened to the floor by dowel-bolts holding down a strap of $\frac{5}{8}$ -inch round iron. The wind-pressure is taken by ties between alternate verticals, and at each end cross walls join the two girders to their full depth.

In August, 1907, when the concrete was a fortnight old, a flood of storm-water, finding its free passage obstructed by the scaffolding, rose over the floor of the bridge without causing the slightest damage. The work was tested with three locomotives weighing 78·6 tons, covering 62 feet of the span from the first to the ninth bay. In the most unfavourable position the deflection did not reach 0·2 inch, and it disappeared completely when the load was removed. The usual speed of trains of 22 miles per hour gives a deflection of 0·14 inch. By designing the bridge with overhanging footways, the dimensions of the piers were reduced to 7 feet 3 inches by 4 feet.

C. A. F.

Stadhouders Bridge at The Hague. Y. M. D. KENTIE.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 848-60 and 875-80.)

The design, construction and testing of the Stadhouders bridge, having a clear span of 78 feet, are described. The material used was overburnt blue bricks in preference to reinforced concrete which did not lend itself so well to decorative work. The calculations for the brick arch were based on the theory of the elastic arch. The arch was built up on one large timber centering which was loaded with the total number of bricks to be used for the arch previous to their being placed in position to prevent excessive settling.

The stresses in the outer fibres set up by variations of temperature (72° F. to 32° F.) are tabulated; it was found that these variations caused a considerable tension in the top outer fibres during cold

weather, so that in order to prevent contraction the bridge was built during the coldest month of the year. The stresses set up by differences in the humidity of the atmosphere were also calculated, and showed the necessity of protecting the brickwork as far as possible against the influence of weather.

The tests were carried out to ascertain whether the assumption that the elasticity-modulus of the brickwork remains constant for different stresses is true. Test columns of the brickwork were subjected to compressive and combined compressive and bending tests, the deflections being measured by means of the Okhuizen extensometer.¹ It was found that the modulus E did not remain constant, thus at a stress of :

9 lbs. per square inch	. . .	$E = 3,000,000$ lbs. per square inch.
210 " " "	. . .	$E = 2,100,000$ " " "
98 " " "	. . .	$E = 2,400,000$ " " "

In the light of these tests it was possible to recalculate the probable deflection of the bridge when loaded. These new values agreed closely with the actual deflection obtained and are graphically compared with it.

H. C. A. T.

*Tests of Nickel-Steel Models of Compression-Members of the New Quebec Bridge.*²

(Engineering Record, New York, 1910, vol. 62, pp. 564-8.)

Eight pairs of models of the bottom chord pieces, diagonal struts, and panel-point connections were made to scales of 2 : 9, 11 : 32, and 1 : 4, mostly the latter, of the full-size members, and tested to destruction. Except the soft carbon-steel rivets, all the material was nickel steel. Types of models are first described, followed by methods of testing, material, records, results, and summary of logs in great detail. There are many illustrations, one being a half diagram elevation of the bridge, giving a key to the members referred to in the text. There is also a table of results. At p. 561 of the same volume there is an account of a test of a model of the carbon-steel chord, the failure of which caused the destruction of the first bridge.

C. O. B.

¹ See Minutes of Proceedings Inst. C.E., vol. clxxvii, p. 375.

² *Ibid*, vol. clxxxiii, p. 14.

Vancouver-Portland Bridges. R. MODJESKI.

(Report¹ to the Presidents of the Northern Pacific and the Spokane, Portland and Seattle Railway Companies.)

This is a description dated 1st December, 1910, by the Chief Engineer, of a series of completed railway-bridges over and between the Columbia and Willamette rivers and connecting the towns of Vancouver and Portland. The former crossing may be divided into three parts, viz., the Washington Channel bridge, Shaw's Island viaduct and the Oregon Slough bridge. The first-named, on the north-east or Vancouver shore, has one 189-foot 6-inch through span, one 466-foot 10-inch drawspan, one 374-foot 8-inch through fixed span, six 269-foot through fixed spans, and one 162-foot through fixed span, these being approximate distances centre to centre. The north abutment is founded on gravel in open excavation, and the masonry piers were founded by the pneumatic process, the four northerly piers on gravel, and the remainder on sand, the maximum depth of which latter was fixed at 80 feet, where the pressure was not to exceed 4 tons per square foot. The Shaw's Island viaduct has twenty-six 80-foot centre to centre deck-plate girders, four to each span, on pile-founded concrete piers.

The Oregon Slough bridge has seven fixed through spans, 162 feet each centre to centre, one 332-foot 10-inch drawspan, and one 57-foot through plate-girder span. The piers are similar in character to those of the viaduct.

Between these structures and the Willamette bridge there is a length of embankment and 2,600 feet of trestle-work, the latter being temporarily provided as additional waterway until a maximum flood should decide how much, if any of it, should be replaced by permanent openings.

The Willamette bridge is symmetrical as regards the 521-foot drawspan, which is the longest in the world. On either side are two 269-foot fixed spans, and at each end is an 80-foot deck-plate-girder approach-span. The abutments are founded on piles and the piers by the pneumatic process. The bridges are for a double line with a net horizontal and vertical distance (from base of rail) of 27 feet 6 inches and 22 feet 6 inches respectively, the headway over the highest flood known being 6 feet in the case of the two northern bridges, but 22 feet 8 inches in the Willamette structure, owing to the fact that the passage of small craft can thus be effected without opening the drawspan. The superstructure is designed for the heaviest modern loading, and the quality of steel used is better than that ordinarily applied to railway structures, the extra strength arising from this cause, and the high-class workmanship, being said to amount to at least 20 per cent.

The piers of the two larger bridges are separately described. The body of the Report ends with a list of quantities. There are five

¹ A copy of this report is in the Library of The Inst. C.E.

appendixes :—(A) List of Engineers and their Assistants and Contractors ; (B) Act of Congress authorizing Construction ; (C) Specification for Superstructure ; (D) Specification for Ashlar ; and (E) Specification for Cement.

The plates, fifty-eight in number, cover every detail of the work, while the frontispiece gives reproductions of photographs of the Washington Channel and Willamette bridges.

C. O. B.

Long Railway-Tunnels (Alpine Tunnels). HEINE.

(Bulletin of the International Railway Congress, London and Brussels, 1910, vol. xxiv, pp. 3491-3551.)

This question having been already dealt with¹ the Author confines himself to supplementary information, but gives details of the construction of the Bosruck and Great Hartberg tunnels. The general method in the Alpine tunnels of the second railway to Triest is to drive a bottom heading as an advance heading ; the Belgian method was, however, adopted in some parts of the Bosruck tunnel. Both this and the Great Hartberg tunnels are single-line. The Bosruck tunnel is 2 miles 1,692 yards long, and the centre-line is straight except at the northern entrance, where it has a radius of $12\frac{1}{2}$ chains. A great part of the tunnel is through triassic limestone, and work was much impeded by inrushes of water. In the Great Hartberg tunnel, which is 1 mile 949 yards long, great pressures were encountered in the decomposed gneiss and the tertiary formations. It proved necessary for the timbering to follow immediately on the advance, which greatly interfered with progress. Full details of construction, a table of construction-cost of seven large Alpine tunnels, and a review of machine- and power-installations are given. The Author considers that the modern Austrian method of driving is advantageously applicable to rock of all kinds ; that the Saccardo system of ventilation is the best ; that machine-drilling should be used as much as possible ; that the Brandt drill is only advantageous in very long tunnels through very hard rock ; that electric locomotives should be used in the finished parts, and benzene locomotives in the working section and bottom heading.

W. H. B.

Tunnels of Switzerland.

(Revue Générale des Chemins de fer, Paris, 1911, vol. xxxiv, p. 130.)

There are 415 tunnels in Switzerland with a total length of 103 miles, the longest being the Simplon of about 13 miles. The

¹ See Minutes of Proceedings Inst. C.E., vol. clxxxi, pp. 505-506.

following is a list of the principal tunnels with their length and approximate elevation :—

	Length. Miles.	Elevation. Yards.
Simplon	13·00	771
St. Gothard	9·50	1,262
Lötschberg	9·25	1,354
Mont Cenis	8·50	1,416
Arlberg	6·50	1,432
Ricken	5·50	678
Montier Granges	5·50	596
Tauern	5·50	1,339
Hauenstein	5·00	494
Mont d'Or	4·00	981
Albula	3·75	1,993
Weissenstein	2·50	789

G. H. S.

Application of the Detonating Cord to Public Works.

BARTHÉLEMY.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. ii, pp. 492-500.)

This Paper describes the demolition of the Hollebeck tunnel over the Canal de la Lys. Work was commenced on the canal in 1864, abandoned in 1870, leaving it in an unfinished state, and recommenced in 1886, when the tunnel, which is 348 yards long, was completed, but in 1893 a length of 106 yards collapsed. After a lapse of 17 years the Belgian Government has decided to complete the canal, raising the water-level about 16 feet 6 inches, and dispensing altogether with a tunnel. It was necessary as a preliminary before destroying the arch to shore up the side walls, and it was decided that the arch should be demolished by 2,125 simultaneous blasts, so that the broken material might be small enough to be easily removed. The explosive decided on was melinite, of which a total weight of 7,166 lbs. was required, and a detonating cord was advised in preference to electric firing, which would have necessitated a very lengthy and dangerous operation. A detonating cord consists essentially of an explosive of great velocity of detonation contained in a metal casing. Particulars of several types are given. The Lheure trinitrotoluene cord of Messrs. Davey, Bickford, Smith and Co. was adopted. A central cord was laid along the axis of the tunnel, from which lateral cords passed to the bottom of each mine. Each end of the central cord was provided with a detonator. Full details of the arrangements and result, which was perfectly satisfactory, are given.

W. H. B.

Electric Equipment of the Detroit River Tunnel.

(Electric Railway Journal, New York, 1911, vol. xxxvii, pp. 62-9 and 104-8.)

These two articles are devoted exclusively to the electrical work of this tunnel. The improvements covered by the electrical installation comprise the reconstruction of two new depots, each with extensive yard-accommodation at each side of the river, as well as the connection between them by the tunnel and its approaches. Besides these, there are a sub-station, two inspection-sheds for the electric locomotives, and five signal-towers, used also as circuit-breaker houses for the third-rail feeders. Having in view the ultimate sharing in the use of the tunnel by other lines than that of the constructors, the Michigan Central Company, its total capacity is determined in the first place. Then, premising that the sub-station is supplied with 60-cycle three-phase current at 4,400 volts from the Detroit Edison Company, the means of supply through any failure being also referred to, the sub-station itself, with its equipment, is described. Descriptions of the third rail and the locomotives complete the first article. The second deals with various auxiliary features, such as the pumping equipment, lighting system, tunnel-electrification details, and the regulation of the load.

C. O. B.

Logging-Railways of the Pacific North-West.

(Railway Age Gazette, New York, Chicago, 1910, vol. xlix, pp. 836-8.)

These lines, aggregating now about 1,000 miles, and increasing to the extent of about 100 miles annually, are of great service to the timber industry. Some penetrate into the heart of the forests for lengths of over 50 miles. They are all of standard gauge and worked by special rolling-stock, light-g geared locomotives being in common but not exclusive use, as $3\frac{1}{2}$ -chain curves and 1-in-6.6 gradients are not unusual, though 1-in-25 is the general limit. The main lines are generally ballasted, and 56-lb. rails are those most in use with sleepers 2 feet apart on centres. The logs are sometimes dragged direct by the locomotive on skids laid between the tracks, but generally trucks are used, the trains weighing often as much as 600 tons, the loads, however, being as a rule with the gradient. Branches are located at a minimum of 1,000 feet apart, due to the economical limit of haul to them through the bush. The cost of a representative main line is given at £1,600 per mile, that of five subdivisions amounting to a similar sum.

C. O. B.

Railways of Servia—1910.

(Revue Générale des Chemins de fer, Paris, 1911, vol. xxxiv, pp. 132-7.)

Servia until quite recently has been much behind the other Balkan States in the mileage of her railways, having at the end of 1907 only 385 miles, 50 of which were of narrow gauge. The present ratio is $1\frac{1}{2}$ mile to every 10,000 inhabitants, against 3 and $2\frac{1}{2}$ in Roumania and Bulgaria respectively, while Greece has $3\frac{1}{2}$ miles.

The principal lines in course of construction, or contemplated, will connect the main line between Belgrade and Sophia, via Nish, with the Bosnian-frontier railways, and the Danube at Prahova, via Nish, with an Adriatic port. The latter line has been under consideration for years, and will be of great importance to the country, whose wealth consists chiefly in cattle. Owing, however, to restrictions recently imposed by Austria, this trade, which, in common with most of the trade of the country, passes through Belgrade, has languished, and has only to a small extent been replaced by one in meat. The other trades are chiefly in fruit and eggs, for the country grows only sufficient corn for home-consumption, and, although at one time covered with forest, now has to import timber. The other imports are chiefly manufactured goods. Hence the importance of an outlet to the sea, which, besides rendering the country less dependent on Belgrade, will open up a fertile country and new centres of commerce and industry. The article contains Tables from which it appears that the number of passengers carried on the railways has increased in 7 years by 233 per cent., and the tonnage of goods by 198 per cent.

G. H. S.

Railways and Tramways of Switzerland—1910.

(Revue Générale des Chemins de fer, Paris, 1911, vol. xxxiv, pp. 117-20.)

The total length of railways in Switzerland in June, 1910, was 2,780 miles, of which about 39 miles were worked by foreign companies. This mileage is composed of ordinary-gauge 2,106 miles, narrow-gauge 588 miles, and rack-railway 86 miles; with 416 miles of double track. The total length of track, including sidings, etc., was 4,242 miles, about half having timber and half steel sleepers. The gradients are often heavy on the ordinary- and narrow-gauge lines. Of the 1,364 stations, that of the Mer de Glace on the Jungfrau Railway is at the highest elevation, viz., 10,410 feet; the lowest is Reazzino, 672 feet.

The tramways are owned by thirty-seven companies and have a total length of 258 miles.

G. H. S.

Tasmanian Railways—1910.

(Report of the General Manager for the year ending 30th June, 1910.)

Average mileage open—	1910	1909
3-foot 6-inch gauge	450·25 }	Not given.
2-foot gauge	23·25 }	
	£	£
Earnings	284,063	280,036
Working-expenses	211,677	204,127
Net revenue	72,386	75,909
Percentage of working-expenses to gross receipts	74·51	72·89
Interest on capital	1·78	Not given.
Passenger-journeys	1,650,455	1,547,016
Goods-tonnage	422,793	467,417
Live-stock tonnage	16,763	15,879
	£ d.	£ d.
Receipts per open and train-mile . . .	599·92 64·32	595·50 65·30
Expenditure per open and train-mile . .	447·04 47·93	434·08 47·60
Net returns per open and train-mile . .	152·88 16·39	161·42 17·70
Subdivision of working-expenses per open and train-mile—	£ d.	£ d.
Maintenance	135·5 14·5	132·2 14·5
Locomotive	180·4 19·3	171·9 18·8
Traffic	109·6 11·8	109·0 12·0
General	21·5 2·3	20·9 2·3
Total	447·0 47·9	434·0 47·6
	£	£
Train-mileage	1,059,790	1,029,119
Engine-mileage	1,276,437	1,256,114
Average passenger-mileage	20·64	21·02
„ ton-mileage	44·85	36·91
Cost of construction and equipment . .	4,048,416	4,003,839
Do. per mile	8,632	8,647
	Gauge.	
Rolling-stock—	3 ft. 6 in. 2 ft.	
Locomotives	72 7	Not given.
Carriages and vans	186 6	
Wagons	1,593 67	

The Report mentions the near completion of 61-lb.-rail laying on the main lines and the consequent intended adoption of more powerful locomotives, increasing loads by 50 to 100 per cent. The State railways have been running for 38 years without responsibility for the death of a single passenger.

C. O. B.

*Yunnan Railway.*¹ A. LE VERGNIER.

(Le Génie Civil, Paris, 1911, vol. lviii, pp. 237-42.)

The Yunnan metre-gauge railway, opened in April, 1910, extends from the termination of the coast-line at Laokay to Yunnanfou, a distance of 288 miles, and traverses a region extremely rugged in some places and very unhealthy in others. The elevation of Laokay is 292 feet, and from this point the line rises to 5,583 feet in a length of 81 miles, whence it descends to the Amitchou plains, where the level is 3,477 feet, only again to rise to 6,645 feet, where the Yang-Tse valley is entered about 20 miles short of the terminus.

Although the railway was contemplated in the French treaty of 1885 no serious attempt was made to ascertain the best course until 14 years later, and the location then determined, although used for estimating purposes, has been entirely abandoned in favour of one considerably further towards the East, where an easier country is encountered. By this substitution it has also been possible to reduce the length of the line by $4\frac{1}{2}$ miles. The estimates were made on the assumption that the coast-line would be available as far as Laokay for the transport of supplies; but such was not the case, and this fact alone goes far to explain the very large increase in the actual cost over the original estimate of £3,800,000. A great number of viaducts have been necessary, and in settling their design one of the ruling factors was that the pieces must be sufficiently light (176 lbs.) for mule transport. In the typical instance described the individual spans are only 26·2 feet, and each trestle is made of two frames spaced at this distance apart; thus the span between two adjacent trestles is the same as that between the two frames of each.

The difficulty of obtaining reliable native labour was very great, and in the lower parts of the line the moist tropical climate resulted in a high death-rate, notwithstanding a very complete medical service.

I. C. B.

*Making Good a Landslip on the Yunnan Railway.*² DUFOUR.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlviii, pp. 123-9.)

Owing to the extreme ruggedness of the surrounding country the line joining the French frontier to Yunnanfou has necessitated the construction of an unusual number of tunnels, viaducts, retaining-walls, and deep cuttings, while the frequent recurrence of treacherous geological formations has been the cause of much anxiety to those

¹ See following Abstract and Minutes of Proceedings Inst. C.E., vol. clxxxi, p. 504.

² See preceding Abstract.

responsible for the construction and subsequent upkeep. The article deals with the works undertaken to make good a rather extensive landslip which took place near kilometre 94 while the railway was in course of construction.

At the point where this damage occurred the line follows the convex side of an extremely steep hill, and at the exact spot a small gully is encountered. The line was literally bracketed from the slope of the hill, and was supported at its outer side by a retaining-wall 32·8 feet high founded well below the bottom of the gully. This wall had exhibited no sign of movement during several wet seasons, and the accident occurred at a most unlikely time, viz. towards the end of a dry period. The result was to carry away 131·2 lineal feet of track, and as the cleavage took place on a slope well below the foundation of the retaining-wall, this also was included in the portion which disappeared. As a temporary measure, the two ends of the breach having been suitably cut back and strengthened, a new 164-foot span bridge which had arrived from home for use at kilometre 386 was requisitioned, and after much difficulty was placed across the opening. As a permanent remedy, various solutions, including the provision of a tunnel, were considered, and eventually the track was sluiced into the hill to the extent of 14·76 feet; while the ground having been restored to its original form by rubble walling resting at a level of 198·8 feet below formation on a concrete wall, a covering also of concrete 1·64 foot in thickness was laid upon the slope, and was pinned down by groups of light rails embedded in concrete, and taking root in the solid ground below the rubble walling. The pitch of the slope is 1 in 1 and the holding-down members are placed chequerwise 16·4 feet apart.

I. C. B.

Track and Line Construction of Electric Railways.

(Engineering News, New York, 1910, vol. 64, pp. 456-62.)

This article gives particulars in great detail of thirty-five street and interurban lines, aggregating about 5,265 miles. Table I gives length, track arrangements and ballasting; Table II, gradients and curves; Table III, rail and rail-joints; Table IV, sleepers and plates; Table V, crossings and switches; Table VI, pole-equipment; Table VII, line and car-equipment, signals, etc. There are other particulars in the text, such as total annual car-mileage, number of passengers, car-hours, etc.

C. O. B.

Lugano-Tesserete Electric Railway.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 56, pp. 317-19.)

The Lugano-Tesserete narrow-gauge electric railway, 5 miles long, which was opened in July, 1909, possesses some novel features. The line, which is metre gauge, begins close to the railway-station at Lugano, runs along the public road to Sassa, and thence close to the road as far as Cannobio, the remainder being on a special track to Tesserete. About 66 per cent. of the line is straight, the remainder having curves of above 230 feet radius. Polyphase current at 25,000 volts is transmitted from Lugano to Tesserete, and there transformed statically to 500 volts, which feeds rotary converters and generates direct current at 1,000 volts, which is used at this pressure on the overhead wires. Tesserete was chosen for the transforming point as the company supplies current for both power and lighting in the Capriasca and Colla valleys. A battery of 458 cells and 160 ampere-hour capacity for a 1-hour discharge is used to avoid fluctuations on the high-tension line. A booster is used, which has a special type of winding, fully described. The overhead wire is carried by uninsulated hangers, but the supporting wires are themselves provided with porcelain insulators. In the stations the posts are of iron, but elsewhere of wood, which, however, does not enter the ground, but is supported by steel framing set in concrete. The rolling-stock is fully described. The ordinary train consists of an eight-wheeled passenger motor-car, carrying sixty persons, and one four-wheeled trailer-car carrying fifty-two persons, or a goods-wagon, the weight of the train being 38 to 39 tons. Each motor-car is fitted with four 45-HP. motors and the speed attained is about $7\frac{1}{2}$ miles per hour. The total cost of the line, buildings, sub-station and rolling-stock was £52,400, and the results are said to be highly satisfactory.

E. R. D.

*Paris North-South Electric Railway.*¹ A. REISSETT.

(Lumière Électrique, Paris, 1910, vol. 12, pp. 393-403.)

The Author describes and illustrates the electrical equipment of the Underground North-to-South Railway of Paris, running in two tubes under the bed of the Seine. The feeding-system is at 600 volts by an overhead circular wire having two diametral indentations, supported in spans of about 18 metres. The third rail is at a potential of 600 volts. Each train carries two motor-sets, one at the front and the other at the rear, one being fed by the positive conductor, the other by the negative. The balance between these two theoretically suppresses electrolysis. The overhead connection

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxviii p. 382, and vol. clxxxiii, p. 345.

is made by a Vedovelli bow, applied pneumatically. The Sprague-Thomson multiple-unit system is adopted with four 125-HP. motors, grouped in two couples of two motors at each end of the train. A detailed scheme of connections is given. There are two sub-stations with double motor-generators. The lighting and signalling systems are described, a diagrammatic signalling scheme being given. The terminal-station arrangements are also discussed.

J. A. S.

Single-Phase System on the Lyons Suburban Tramways.

O. LOMBARD-GERIN.

(Le Génie Civil, Paris, 1910, vol. lviii, pp. 89-93.)

The Omnibus and Tramway Company of Lyons has from time to time absorbed various small undertakings, and in consequence now controls a great variety of systems where even the gauge is not uniform, only 152 miles out of 186 being standard. The Company has laid down and equipped on the Westinghouse single-phase system two lines radiating from Lyons and ending respectively at Jons and Miribel; these have been in use for over a year, and the article deals not only with the original design, but with the improvements which experience has shown to be desirable, and with the cost of working as compared with that of a neighbouring section operated by continuous current and accommodating a somewhat similar class of traffic. The total length of the respective lines is $12\frac{1}{2}$ and $7\frac{3}{4}$ miles, but a large extension is contemplated in both cases. Until quite clear of Lyons each line is operated by continuous current, and it is only when the country is reached that the alternating system is brought into use. The pressure in the high-tension overhead wires is 6,600 volts, which is transformed down to 275 volts on the car itself. The trolley-wires are of double-headed section, having an area of 0.08 square inch, and are hung from a steel cable by iron stirrups. Considerable trouble has been experienced through short circuits caused especially by birds perching on the stay wires and catching their wings on the supporting brackets, which are earthed and thus in communication with the return. The result of this short-circuiting has been the splitting of a number of insulators by reason of the arcking which was induced, and of course the repeated pulling out of the circuit-breakers. In future extensions the difficulty will be got over by a slight rearrangement. The working-costs are a little in excess of those of the continuous-current section used for comparison, but the conclusion drawn is that such excess would disappear when the length of line was increased to 12 miles, while the lower initial outlay militates in favour of the high-tension system.

I. C. B.

Efficiency of the Steam-Locomotive. DR. SANZIN.

(Zeitschrift des oesterreichischen Ingenieur- und Architekten-Vereines, Vienna, 1910, vol. lxii, pp. 725-31.)

The Paper is divided into four sections dealing with the mechanical and overall efficiency of the locomotive, and the thermal efficiency of the engine and boiler respectively. Tests were carried out on a number of passenger- and goods-locomotives. The mechanical efficiency, defined as the ratio between the tractive effort at the wheel-tire and the indicated effort, varies with the cut-off, being 0·64 at 10 per cent. and 0·91 at 35 per cent. cut-off for a locomotive-speed of 25 miles per hour. The influence of speed is not so marked, the mechanical efficiency being 0·95 at 6·25 miles per hour, and 0·87 at 32 miles per hour with a cut-off of 30 per cent. in each case. The overall efficiency is taken as the ratio of the drawbar-pull to the indicated effort. In this calculation the observations become more complicated, since the efficiency depends on the weight of the locomotive and tender ($L + T$), the extra effort, due to a rise or fall of the track, $\pm i$ pounds per ton, and the resistance due to change in velocity, $\pm b$ pounds per ton, so that if Zd be the drawbar-pull and Z_i the indicated effort, the efficiency

$$\eta_1 = \frac{Zd - (\pm i \pm b) (L + T)}{Z_i}.$$

b is generally taken as $360 p$, where p is the change in velocity in feet per second.

A Table comparing the overall efficiencies of a two-cylinder 2B1, a four-cylinder 2B1 and a four-cylinder 1C1 express locomotive is given.¹

Speed. Miles per Hour.	Compound Locomotives.		
	Two-Cylinder 2B.	Four-Cylinder 2B1.	Four-Cylinder 1C1.
25·0	0·87	0·86	0·91
31·0	0·84	0·86	0·91
37·5	0·80	0·86	0·86
43·5	0·73	0·85	0·82
50·0	0·65	0·81	0·78
56·0	0·54	0·75	0·72
62·0	0·4	0·68	0·63
67·5	..	0·58	0·52
75·0	..	0·44	0·36

¹ The letters A, B, C, etc., indicate the number of coupled driving-axes; 1, 2, 3, etc., respectively, the numerals before and after these letters, giving the number of leading and trailing axes.

The Table shows that for speeds above 37·5 miles per hour the 2B1 and 1C1 locomotives are more efficient ; and at high speeds the 2B1 (Atlantic type) appears to be the best.

The Author next considered the effect of gradients on the overall-efficiency and tabulated the results of a test with a 1C1 compound express locomotive. Tables showing the effect of change in gradient, from which the most suitable type of locomotive and the train-load for various sections of a railway may be deduced, are also given. Results with compound goods-locomotives are tabulated in a similar manner.

C. J. G.

Three-Years' Experience with Superheated-Steam Locomotives.

J. T. BIERMANN.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 634-41.)

Comparative tests, extending over 3 years, of superheated-steam locomotives were made by the Holland Railway Company. The tabulated results may be summarized as follows :—(1) The coal-consumption is 17·5 per cent. less than with the older type of saturated-steam locomotive ; (2) The water-consumption is 20 per cent. less ; (3) The capacity of the locomotives has been increased so that for the same speeds the weight of the trains can be increased by 25 per cent. or the same weight can be propelled at a greatly increased speed ; (4) The boilers are not forced to such an extent since about 20 per cent. less coal is burnt per square foot of grate-area per hour, thus ensuring longer life to the copper fire-boxes ; (5) The cost of upkeep of the locomotives has not been increased ; (6) The primary cost has been increased by only a small amount. Indicator-diagrams, continuously taken on some of these engines, are reproduced.

H. C. A. T.

Belgian Locomotives.

(La Technique Moderne, Paris, 1910, vol. ii, pp. 597-601)

Recently three types—Nos. 9, 10 and 36—of heavy locomotives have been adopted by the Belgian State Railways. No. 9 is of the 4-6-0 type, designed for fast traffic on level roads ; No. 10 is a "Pacific" type for fast passenger-traffic on hilly roads ; and No. 36 is a five-coupled goods-locomotive. In all cases the pressure is 14 kilograms per square millimetre and the steam is superheated by a Schmidt superheater, Fournier thermometers being provided for measuring the temperature. Each engine has four simple cylinders, No. 9 of 445 millimetres diameter, 640 millimetres stroke ; No. 10 of 500 millimetres diameter and 660 millimetres stroke ; and

No. 36 of 500 millimetres diameter and 600 millimetres stroke. Piston valves are used throughout. The weights and tractive effort are as follows:—No. 9, 82 tons, 11,650 kilograms; No. 10, 102 tons, 15,166 kilograms; and No. 36, 104 tons, 20,700 kilograms. The boiler-tubes are of brass—50 millimetres external diameter—except those containing the superheater-tubes, which are of steel, 127 millimetres external diameter. A Plate gives several fully-dimensioned sections of type 36.

H. R. S.

Goods-Locomotives for the Norwegian Trunk Railway.

O. STORSAND.

(Teknisk Ugeblad, Christiania, 1910, vol. 57, pp. 570-71.)

The Paper is accompanied by a photographic view of the engine and tender and by drawings. The locomotive has four coupled axles, each with a load of 14·3 tons, and a leading radiating axle with a load of 10 tons. The latter has a movement of 2 inches to each side, and the second and fourth coupled axles a parallel lateral movement of $\frac{1}{4}$ -inch to each side, which enables the engine to pass over 400-foot-radius curves at low speeds. There are two cylinders, 22 inches in diameter by 25 inches stroke, working with superheated steam at a pressure of 170 lbs. per square inch and a temperature of 570° to 660° F. The boiler has a heating-surface of 1,460 square feet and a superheating-surface of 432 square feet. The grate-area is 28·2 square feet. All rotating masses are perfectly balanced and reciprocating parts to the extent of 42 per cent., which, at a speed of 30 miles per hour, alters the loads on the coupled wheels by ± 15 per cent. The superheater has twenty-four elements, consisting of a $1\frac{1}{4}$ - $1\frac{1}{2}$ -inch tube, bent double inside a fire-tube of $4\frac{1}{8}$ - $5\frac{1}{4}$ inches diameter, and suitably connected to cast-iron headers in the smoke-box. To prevent the superheater-tubes from being burned there are dampers which automatically close the large tubes when steam is shut off and open them again on re-admission of steam. The dampers can also be worked by hand from the foot-plate. The slide-valves are circular, and the motion is of the ordinary German type overhung and actuated from the outside crank-pins. When running without steam a circulating-valve between the two cylinders is brought into action by hand, and valves admit air into the cylinders, thereby creating a slight air-exhaust sufficient to prevent soot and cinders from dropping into the steam-chest. Each cylinder is also fitted with a safety-valve and the usual drain-cocks. The piston-rods are continued forward through the front cylinder-covers and work in sleeves, which, together with the cross-heads, carry the weight of the pistons and rods, the packings of the stuffing-boxes being allowed a slight play to accommodate themselves to the rods. The locomotive is fitted

with a combined steam and hand screw-brake acting through equalizing-gear on brake-blocks on each of the coupled wheels. The tender has a screw hand-brake acting on two brake-blocks on each wheel.

L. S. Z.

Comparative Locomotive-Tests on the Italian State Railways.

BOSHART.

(Organ für die Fortschritte des Eisenbahnwesens, Wiesbaden, 1910, vol. xlvii, pp. 378-91.)

These tests were carried out on the express- and goods-engines used on the Pennine section from Pistoia to Pracchia, the average rise of which is 1 in 45, with about 50 per cent. of curves. Some twenty-two locomotives of German and American manufacture were tested and gradually eliminated until their number was reduced to three. These were subjected to lengthy tests, the results being slightly in favour of the type using superheated steam. The following Table gives the results obtained :—

	Ordinary Tubes.	Corrugated Tubes.	Superheated Steam.
Type of locomotive	630		640
Weight without tender . . . tons	55	55	54·5
Heating-surface . . . square feet	1,315	1,880	1,160
Grate-area : heating-surface	1 : 52	1 : 72	1 : 45
Maximum HP.	800	860	950
Indicated HP. per ton	14·6	15·6	17·4
Indicated HP. per square foot of heating-surface	0·6	0·45	0·81
HP. as compared with standard engine	1·06	1·15	1·27
Train-load tons	192 to	305	218 to 319
Coal-consumption, pounds per effective HP.-hour	0·97	„ 1·26	0·86 „ 1·26
Coal-consumption, pounds per indicated HP.-hour	0·6	„ 0·845	0·56 „ 0·77
Coal-consumption, pounds per ton-mile	0·068	„ 0·1	0·056 „ 0·775

The results obtained from the remaining locomotives tested are also tabulated.

C. J. G.

Swiss Normal-Gauge Electric Locomotives.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 58, pp. 247-51.)

This article gives a short history of the evolution of the electric locomotive as now used on the normal-gauge railways in Switzerland. The Author traces changes and improvements in design, and describes the chief features of the leading varieties, giving an outline diagram of each type with chief dimensions and weights. The earlier designs were intended solely for the narrow-gauge railways, which are so common in Switzerland, but their success caused designs to be invited for ordinary-gauge lines. The type which has finally been evolved is characterized by the rigid fixing of the electric motors in the frame of the vehicle, and by the driving through connecting-rods on to the wheels either from the motor-shaft or from an intermediate shaft geared to the motor-shaft, or driven from it by cranks and connecting-rods; recently the use of gearing has been abandoned. The following are extracts from detailed descriptions in the original article:—On the Burgdorf-Thun line two types of locomotives are illustrated, that of 1899, with four wheels, weighing 30 tons, and that of 1910, with eight wheels coupled in two sets, weighing 42 tons, and supplied with polyphase current from the overhead line at 750 volts and 40 periods. On the Seebach-Wettingen line single-phase current at 15,000 volts and 16 periods is used; the locomotives weigh 40 tons and have eight wheels coupled in two sets. The first built had a transformer to change the alternating current into direct current, but the second type transformed into single-phase at 350 volts, and had gearing and an intermediate shaft. For the Simplon line the 1906 design had six coupled wheels and a two-wheeled bogie at each end and weighed 44 tons, but the 1907 type had eight wheels all coupled and weighed 68 tons. In both no intermediate shaft was used, but crank-driving with connecting-rods. Energy was supplied by three-phase current at 3,000 volts and 15 periods. The Bernese-Alps Railway uses 15,000 volts and 15 periods on the overhead wire. The first type of locomotive had twelve wheels coupled in two sets and weighed 86 tons, and the second type had eight wheels coupled in two sets and a two-wheeled bogie at each end and weighed 93 tons. On the Wiesenenthal Railway a passenger-train locomotive built in 1910 has six wheels coupled and a two-wheeled bogie at each end, with 10,000 volts and 15 periods on the overhead wire and 1,000 volts at the motors, and weighs 65 tons. For the Southern Railway of France two types have been built, one by the Swiss Locomotive Works, in conjunction with Brown, Boveri and Co., which has six wheels coupled and a two-wheeled bogie at each end and weighs 80 tons; the other in conjunction with the Thomson-Houston Co., of Paris, which weighs 86 tons. The Dessau-Bittersfeld Railway has locomotives by Brown, Boveri and Co. which have eight wheels coupled, weigh 56 tons and have 10,000 volts on the overhead wire. Lastly, the

Paris-Lyons-Mediterranean Railway uses locomotives, built by E. G. Alioth, which have two separate units of motors, with two sets of four wheels coupled and a four-wheeled bogie at each end. These are for express trains and weigh 136 tons; they receive single-phase current at 13,000 volts and 25 periods.

E. R. D.

Invalid-Coaches on Swiss Railways.

(Revue Générale des Chemins de fer, Paris, 1911, vol. xxxiv, pp. 140-2.)

Four coaches have recently been placed on the Swiss Railways for through conveyance of invalids. They are double-bogie carriages about 62 feet 9 inches long and weighing 42 tons, fitted with Westinghouse automatic and Hardy vacuum-brakes, and otherwise arranged to suit the French, German, and Austrian trains to which they may be attached. They are divided into eight compartments, viz., one for invalid, two for ordinary passengers, one for doctor, two lavatories, one kitchen, and one for luggage. There is a side corridor with platform at each end, where there are entrances, besides which there is a central door leading directly to the invalid's compartment. This and the adjoining lavatory are arranged as in a private hospital, and fitted with electric bed-warmer, fans and lamps, and supplied with water heated by electricity to a constant temperature. The doctor's and passengers' compartments have sleeping accommodation. The kitchen is arranged on the most approved principles, and all cooking is done by electricity. The general heating of the carriage is by steam with radiators, which in the case of the invalid's room are regulated to keep it at an even temperature. The lighting of the kitchen is effected by means of a dynamo worked from the axle, which charges eight batteries of accumulators.

G. H. S.

Fire-Protection in the New York Station of the Pennsylvania Railroad.

(Railway and Engineering Review, Chicago, 1910, vol. 1, pp. 1000-2.)

The area of this new station¹ is 38 acres, with three levels below the main floor, the lowest being 36 feet below the street. There are 3 miles of piping, weighing 425 tons. The means by which provision is made to secure a continuous supply of water meeting all demands for domestic service, and a surplus sufficient to maintain not less than twelve standard fire-streams, or 3,000 gallons per minute, are described in detail. The connections with the street-

¹ See Minutes of Proceedings Inst. C.E., vol. clxxix, p. 397.

mains and the method by which any serious failure in one or another can be met, are referred to. The tanks and pumps fully described are in a separate building. Hand chemical extinguishers in the upper corridors and others of the non-freezing type in the lower part are used in addition. The fire-brigade organization is then described, and the article closes with an account of the alarm system, with tunnel-alarms for transmitting to near and distant points signals indicating fire, and for cutting-off current to the power-rails. An article on this subject also appears in *Engineering News*, New York, 1911, vol. lxiv, p. 472.

C. O. B.

Snow-Sheds on the Great Northern Railroad, U.S.A.

(*Railway Age Gazette*, New York and Chicago, 1911, vol. 1, pp. 82-5.)

The shed more particularly described was erected in consequence of two trains and seven locomotives having been struck by a snow-slide near the mouth of the Cascade tunnel, Washington, resulting in much loss of life and property; and its construction by the novel means of reinforced concrete was the result of a Report of a Committee of the American Railway Engineering and Maintenance of Way Association. The reinforced-concrete shed is of three types, the principle common to all being that the roof corresponds as much as possible to the original slope of the ground before the cutting was made, the variation consisting in the main support of the shed next the mountain side of the formation, which is adapted for a vertical rock-cut, or to act as a retaining-wall either partially or wholly. Full details are given, including the arrangement of carrying some of the reinforcing of the rafters from the ridge downwards into the rock in which they are anchored, thus resisting the stress communicated through friction by the sliding snow. The roof-slabs are designed for a load of 1,100 lbs. per square foot, so as to include weight of stones and trees carried down. The shed, which is 3,300 feet long and cost £96,873, is also described in *Engineering News*, New York, 1911, vol. lxiv, pp. 652-3, and in *Engineering Record*, New York, 1911, vol. lxii, p. 729.

C. O. B.

New Form of Brick-Kiln. Dr. W. WOLLENWEBER.

(*Glückauf*, Essen, 1910, pp. 1706-9.)

In the Rhenish-Westphalian coal-mines, where the argillaceous shale seams are utilized for brick-making, now an important and growing industry connected with coal-getting, one system is almost exclusively followed. The hard shale is first reduced to a state of fine division, then moistened with 5 to 8 per cent. of water, pressed

in moulds, and burnt in the Hoffmann annular kiln. The invention of this kiln may be said to have given birth to the subsidiary industry under consideration. The principal features of the Hoffmann kiln are continuous action, and utilization of the spent heat to warm the ingoing charge and the air-supply to the furnaces. The new "canal" or straight-flue kiln of Dr. Möller and Professor Pfeifer, which has recently come into competition with the Hoffmann kiln, retains all the advantages of the annular form, i.e., it utilizes the unused heat of the kiln to warm the ingoing charge, and the heat given off by the out-coming charge to warm the air for combustion, and offers greater facilities for a rapid output, with other advantages due to the single straight "canal" in which the preliminary heating, the burning and the cooling zones are always in the same place while the charge is passing through on wheeled trucks. The chief difficulty encountered—that of protecting the iron parts of the trucks from the destructive effects of the hot gases—has been overcome by separating completely the upper portion of the space, the kiln proper, from the lower in which are the axles, wheels and truck-floors. The trucks are loaded with the moulded bricks at the mouth of the kiln and then run in, pushing those in front forward, the kiln being always full of loaded trucks. Each fresh truck run in forces the whole train forward the length of one truck, and a sufficiently burnt load is driven out at the other end. The axles and wheels of the truck carry above them an iron platform on which rests a thick floor of refractory material. When these platforms come together end to end in the kiln, they form a continuous floor, which is the partition for the protection of the iron parts. The side edges are provided with vertical strips of iron, which dip into sand held in troughs on the sides of the kiln, thereby making a practically gas-tight connection. Sectional views are given to show the construction, and also the method of bringing in the cold air for combustion through the exit end of the kiln and over the portion of the train to be cooled for the purpose of raising the temperature of the air. The Author states that the output of this form of kiln is four times greater than that of the annular form. He mentions works where the daily output of bricks is 40,000. This straight kiln is working satisfactorily at several mines, notably at Hamm and at Rauxel.

G. G. A.

Danger of employing Iron in Masonry Structures. G. DENIL.

(Annales des Travaux Publics de Belgique, Brussels, 1910, vol. xv, pp. 921-1010.)

In the great majority of cases where iron has been leaded into stone, cracks have sooner or later made their appearance; these are due to the swelling caused by the oxidizing of the metal, and the only way of avoiding them is to make a tight joint between the lead and the iron. The lead in no sense adheres to the iron, thus

tightness can only be secured by efficient caulking, and this can only be accomplished when (a) the section of the metallic member approaches a circle, (b) the hole in the stone takes the same form, (c) iron fixing wedges are eliminated, (d) no projections in the iron such as those caused by "ragging" exist, and (e) the lead is reasonably pure, a state which makes for ductility. A sketch of an ideal bolt-attachment is given, and in this the inner end of the bolt is swelled, so that the portion within the stone is conical in form, while, to prevent turning when the nut is being screwed on, four longitudinal grooves are provided, into which the lead will readily flow and thus form a key. Galvanizing is recommended, but, if absolute immunity from cracking is called for, the metal within the stone should take the form of an elongated bronze nut, into which can be threaded an ordinary iron bolt. The employment of bronze in this manner does not add more than 33 per cent. to the cost of the attachment, while it may effect a saving by the possible diminution in the size of the stone. The Author is acquainted only with a single case where iron members fixed by means of cement have been sufficiently long in place to give a fair test, viz., a railing on the right bank of the Maus. This railing was fixed in 1880, and every one of the stones into which the uprights enter is now cracked. The article concludes with a very complete discussion on the probable life, under various circumstances, of the armouring in reinforced concrete, and the point is emphasized that any circulation of water in the envelope is fatal. The various preparations on the market for securing impermeability in concrete are mentioned, but the Author regards them as more or less dangerous, and prefers, wherever possible, to provide an independent covering.

I. C. B.

Conical Concrete Piles in Foundation Work. W. SIEGFRIED.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 56, pp. 235-9.)

The piles recently employed for the foundations of the large coal-bunkers at the Bern Gasworks were constructed on what is known as the Stern system, and may be classed as "floating" piles, as they are not driven down to a solid stratum of rock or clay, nor do they depend on skin-friction, as would be the case in piles of equal cross-section throughout their entire length. They act rather as wedges which tend to compress the strata through which they are driven. A conical sheet-iron sheath is fitted with a specially made conical wooden plug, and both are driven together into the ground, the core is then withdrawn and the tube left in the ground and afterwards filled with cement concrete. The drawings show that the pile used at the gasworks had an iron sheath 11.48 feet overall in length, 10 inches internal diameter at the bottom gradually increasing to 11.8 inches at a distance of 6.56 feet up, and thence increasing much more rapidly to 17.7 inches at the top. The wooden

core is an easy fit inside the sheath, and is shod with a steel point. It was decided to allow a load of 30 tons per pile, and the total load is carried on fifteen groups of twelve piles each, each group consisting of two rows of six piles each. The ground, which, was made up of rubbish, gravel and filling, was excavated 11.48 feet deep until the surface of the ground-water was reached, and then the piles were driven. The water rose to the top of the tubes, and the concrete was then carefully filled in to each sheath down a long funnel. Reinforcement consisting of steel rods was inserted some feet into the top of the piles, and each rod was bent over at the top and the whole embedded in the concrete block which covered each group of twelve piles. About twenty piles were filled per day, and the entire work of piling and filling with concrete was completed in 21 days. The cost is said to be much less than that of any other system.

E. R. D.

Influence of Fineness of Cement on its Tensile and Compressive Strength. M. E. H. TJADEN.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 986-8.)

The hardening of cement, after its admixture with water, depends on the chemical reaction of the latter on certain constituents of the former, principally tri-calcium-silicate and bi-calcium-aluminate. It is therefore obvious that the finer the grain of the cement the more readily will the particles of water penetrate to these salts, so that the cement will harden more rapidly. In order to investigate the truth of this reasoning more minutely, the Author undertook a series of experiments with three different grades of cement. These various grades were subjected to compressive and tensile tests after 7 and 28 days' hardening under water at a constant temperature of 68° F. The results of these tests, recorded in Tables and represented in graphs, show that the relative compressive strength of the fine and the coarse cement varied between ratios of 7 : 1 and 11 : 1 after 7 days, and of 4 : 1 and 5 : 1 after 28 days' hardening. The Author recommends, therefore, that a residue not exceeding 3 per cent. and 20 per cent. for a 5,660-mesh and a 30,800-mesh (per square inch) sieve respectively, should be stipulated in all specifications dealing with cements.

H. C. A. T.

Le Chatelier Boiling-Test of Portland Cement.

D. B. BUTLER, Assoc. M. Inst. C.E.

(Transactions and Notes, Concrete Institute, London, 1910, vol. ii, (Pt. 2), pp. 89-115.)

Though, with improved methods of manufacturing cement, cases of complete disintegration are now rare, expansion of a more or less dangerous nature is not infrequently met with. The Author refers

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to the earlier tests for soundness and to the chief methods introduced at later periods for testing expansion; namely, those of Faija, Deval, the boiling-test and the Erdmenger and Heintzel tests, also to the Bauschinger and the Le Chatelier apparatus for the accurate measurement of minute alterations in the volume of cement samples. The description of the latter apparatus, taken from the British standard specification of 1907, is reproduced. It is shown that different operators with the Le Chatelier apparatus are liable to obtain inconsistent results. It is stated that one cause of this variation is the difference in the elasticity of the metal moulds, and an illustration is appended of a calliper device for determining the elasticity of the metal ring, with comparative results of tests of brass and nickel moulds; the latter being in certain instances extremely stiff, as contrasted with the brass moulds. Some other causes for discrepancies in the results by this test are discussed, and the observation is made that there is frequently less expansion on boiling than in cold water. Facts relating to aeration and to the time occupied in hardening are likewise considered, and some tests are given to explain the action of fine and coarse particles; the grit showing enormous expansion as compared with the original cement. The Author maintains that the outcome of the facts above recorded would seem to indicate that the results of the Le Chatelier boiling-test should not be too rigidly interpreted. Attention is called to the abandonment of the accelerated tests by German authorities, and it is stated that considerably less than 20 years ago not 10 per cent. of the Portland cement manufactured in England or elsewhere would withstand boiling water under the foregoing conditions. In the discussion on the Paper instances were given of the inaccuracies and uncertainties of the Le Chatelier test by Mr. H. K. G. Bamber, Mr. W. J. Cooper, Mr. R. Gordon Nicol, Mr. Watson and others.

G. R. R.

Loftiest Concrete Chimney in the World. JACQUES RABUT.

(La Nature, Paris, 1911, Premier Semestre, pp. 44-5.)

It is pointed out that, while in France confidence in the use of reinforced concrete for chimney-construction had not yet made much headway, this material has been employed for many years in the United States for chimneys of the largest size. The structure here described and illustrated has been erected for the Colusa Parrot Mining Company in the State of Montana. The height is 350 feet and the internal diameter 17·7 feet. It is of the type known as the "Weber Cylindrical Chimney," and is formed of two entirely separate cylindrical tubes, placed one within the other, 4 inches apart, so as to afford an annular air-space all the way up. The height of the interior tube varies with the use to which the chimney is to be put, or with the temperature of the flue-gases to be discharged. For boiler-chimneys the lining-tube goes up about one-

third of the height, whilst in the case of furnace-chimneys for ore-smelting the lining is carried up all the way. It is asserted that by this means all the expansion and contraction is taken up by the protected inner tube and the structure is not exposed to dangerous strains. If the inner tube is carried up only a part of the way an expansion-joint is inserted at the point where the double cylinder ceases. The foundations in reinforced concrete are extended so as to spread over a sufficient area of base to take the load safely and the reinforcement of the cylinders consists of vertical and horizontal bars spaced at suitable and regular distances apart. No scaffolding is needed for building the chimney, as the work is all done from the inside, and a shifting mould, formed of wooden panels, is raised in stages as the erection proceeds. It is claimed for chimneys of this type that they are veritable monoliths, without a single joint from top to bottom, are much lighter and stiffer than those of any other material and less exposed to damage from fires than the usual structures. Moreover, as very little plant is required for their erection, they are far cheaper to build in remote and inaccessible districts than the ordinary type of chimney. They can be constructed with great rapidity, and local sand and gravel can be used. It is stated that in the past 10 years some 300 chimneys of this type have been erected, fifty of which were 200 feet in height and upwards, and five over 245 feet high.

G. R. R.

Reinforced-Concrete Coal-Bins. M. S. FALK.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1281-8.)

The construction of two reinforced-concrete coal-bins which have been recently erected for New York firms forms the subject of this Paper. One of these bins is constructed entirely of reinforced concrete, and of the other only the posts and bottom are of concrete, the sides and roof being of timber. The all-concrete bin has a capacity of 3,600 tons. It is 100 by 55 feet over all, and is 60·5 feet high from the ground to the top of the roof. It is carried on twenty-eight reinforced-concrete posts, each 30 inches square, spaced 12 feet 5 inches apart in the clear. The bin overhangs the outside row of columns by 4 feet 6 inches, and is divided into twelve compartments by 8-inch reinforced-concrete partitions. The weight of the coal is carried on a concrete slab 16 inches thick reinforced in two directions by $\frac{7}{8}$ -inch corrugated bars. The coal is delivered to wagons through shoots in the bottom, four to each bin. The method of depositing the concrete by means of a bucket-conveyer extending the full height of the outside walls is described. The other bin has a capacity of 1,430 tons, its dimensions being 62 by 46 feet over all. Owing to the difficulty in depositing concrete as found in the first bin, the second is of concrete only as far as the

foundations and floor. This bin rests on four lines of columns, nine in a row, and spaced 16 feet 8 inches centre to centre. The floor-slab is 6·5 inches deep. The Author states that the second type of bin is more easily erected than the other and is entirely satisfactory.

R. S. B.

Experiments with Reinforced-Concrete Masts. K. ZEBRAK.

(Technische Blätter, Prague, 1910, vol. xlii, pp. 119-34.)

The mast experimented with was 10 metres high and 30 centimetres in diameter at the butt, reducing in four steps to 20 centimetres at the tip. The core was a wooden pole 18·5 centimetres at the butt and 9·5 centimetres at the tip. The reinforcement consisted of fifteen steel rods of 7 millimetres diameter at the butt, reduced to seven rods at the tip, laced together with spiral wire windings and nailed into the wooden core. The concrete surrounded the wooden pole and had an average thickness of 5·5 centimetres. For testing, the mast was laid horizontally and the butt was held in two cast-iron clumps placed about 180 centimetres apart, the tip being pulled over by means of tackle. The deflection was measured at four points along the length of the mast, loads of 140 to 500 kilograms being applied. At the latter load the deflection of the tip of the pole was 38·5 centimetres, and cracks appeared in the concrete at a load of 400 kilograms. The load was then released, and a slight permanent set was observed. On reapplying the load, the deflections were found to be somewhat greater than previously, and at a load of 645 kilograms the mast failed. Elaborate calculations are given and the testing of the pole is illustrated by photographs.

H. R. S.

Manufacture of Reinforced Posts and Hollow Tubes.

JACQUES BOYER.

(La Nature, Paris, 1910, Deuxième Semestre, pp. 394-5.)

This process is based on the effect of centrifugal action exerted on the contents of moulds caused to revolve rapidly. The machine used, which is the invention of Messrs. Otto and Schlosser, is shown by means of a photograph. The moulds, constructed of wood, lined with sheet metal, are arranged horizontally and can be made of any required length. The metallic reinforcement, which consists of round rods of Siemens-Martin steel, placed longitudinally, is kept in position by spiral interlaced windings of steel strip. The distance apart of the rods is adjusted by means of concrete fillets, and when the reinforcement is inserted into the mould the liquid concrete is run in and the mould is placed in the machine. This

is composed of a number of similar units, spaced at suitable distances apart. Each unit consists of two massive cheeks having a circular opening in the centre. Three equidistant shafts capable of being shifted radially, on each of which are fixed two wheels, are arranged concentrically around the central aperture. Between the three pairs of wheels a tubular body, passing through the above-mentioned apertures in the cheeks, is made to revolve. This is provided laterally with regulating clamps similar to those in a lathe-chuck. The whole is driven by pulleys from a single line of shafting, which is parallel to the horizontal axis of the moulds. There is further an arrangement of friction-rollers beneath the machine, the first and last of which are stationary, but the intermediate ones are fixed in a single frame, which can be raised or lowered by means of a lever. The mode of working is as follows: the mould is slid into the tubular body of the machine by the aid of the friction-rollers and it is then firmly clamped between the regulating jaws and centred accurately. Next the frame with the runners is depressed so that the weight of the mould comes on to the driving-pulleys, and it is thus caused to turn at 500 to 1,000 revolutions per minute. By the action of the centrifugal force the liquid concrete is driven against the walls of the mould and the metallic reinforcement becomes rigidly embedded in the mass. The water, owing to its inferior density, is gradually squeezed out and passes towards the hollow centre, while the concrete is compressed against the lining of the mould. After running for 10 or 15 minutes the machine is stopped and the mould, with the tube or mast, is removed. The concrete mass remains 12 to 24 hours in the mould and it is then withdrawn, and covered with moist sand for 3 or 4 weeks to become fully indurated.

G. R. R.

Experiment in Waterproofing Concrete. E. SCHICK.

(Beton und Eisen, Berlin, 1911, vol. x, p. 15.)

In the course of the erection of a large silo for grain and a flour-warehouse adjacent to it, both constructed in reinforced concrete, it was considered necessary to render impervious to water the foundations consisting of a bed of rammed concrete 5·57 to 6·56 feet in thickness. Both buildings situated in the vicinity of Budapest are close to the Danube, which is liable to frequent floods. In order to prevent disturbances to the milling operations, it was deemed important to make the concrete foundations waterproof, and for this purpose the use of a layer of "ceresit" was proposed. The heavy cost entailed by the adoption of this material over a surface of 21,500 square feet led to an attempt to find a substitute. It was ascertained by analysis that the chief active ingredient in the specifics in common use for waterproofing concrete was "potash

soap" (soft soap). On the advice of Professor Zielinski, a layer of fine concrete 4 inches in thickness was introduced over the whole area. The proportions used are specified, and the surface-coat, about one-tenth of this thickness, consisted of a carefully-levelled bed of fine cement-mortar (one part cement to three parts of Danube sand). This bed was closely rammed, and, instead of using pure water to mix the cement and sand, a soap-solution containing about 5 to 7 lbs. of soft soap to each cubic yard of concrete was employed. This layer, in the course of a flood which occurred soon afterwards, proved to be wholly watertight, and it was executed at about one-tenth the cost of using patented materials. Some experiments specially carried out with a tank constructed of the concrete mixture with soft soap are described, and the amount of protection afforded was investigated. Water was found to penetrate the mass less than $\frac{1}{10}$ inch. The Author is of the opinion that the protective action is not due to the repulsive effect of the fats and mineral oils in the soap, but rather to the combination of the free lime in the cement with the fatty acids. The lime expels the potash from its compounds and forms insoluble salts which close up the pores in the concrete.

G. R. R.

Subaqueous Concrete Work. H. R. LORDLY.

(Transactions of the Canadian Society of Civil Engineers, Montreal, 1910, vol. xxiv, pp. 122-50.)

The Author describes the construction of the extensive subaqueous concrete work in connection with the Quebec Canal, and more particularly the construction of the 5 miles of concrete wall, without unwatering, on the Lachine Canal. The banks of the Lachine Canal were originally pitched with stone excavated from the work. Some of this stone disintegrated under wave action and large areas of the walls were destroyed. An attempt was made to rebuild the walls in heavy courses, but, as work could only be carried on for two or three weeks in the year during the period of unwatering, a more expeditious method had to be used. The face of the slope-wall is about 24 feet, the slope being 1 to 1, and it was decided to construct these slopes of concrete while water was in the canal. The concrete was laid 18 inches thick, and as there was danger of it clogging between the mould-boards and the projections of the old masonry wall, and thus creating large voids, a ram was designed to force the concrete down between the mould-boards and the prepared slope. The ram was made of timber, and the guide of 15-inch steel channels. The ram was dropped by gravity. The velocity of water in the canal was about 1 mile per hour. The moulds were 100 by 22 feet face, and the plant was designed for 200 yards per day. The moulds were floated to the work, placed by the aid of a lifting-scow and divers, and heavily loaded at the bottom. The scow had to be made very narrow so as not to

obstruct navigation. The Author describes the apparatus and plant in detail, and the slight defects noticed in the work when the canal was drained in the spring. The work done late in the autumn was not so good as that done earlier. The Paper is illustrated by photographs and drawings.

R. S. B.

Bond-Friction-Resistance in Reinforced Concrete. W. F. SCOTT.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1244-80.)

The object of this investigation was to show that the usual assumption that reinforced concrete is practically a homogeneous material is not true, and it follows from this that sectional planes before bending do not remain plane surfaces after bending. The first phenomena noted were: (1) That perfect bond between the concrete and the surface of the steel did not seem to be lasting; (2) After the real bond was broken there was a creeping of the rods, accompanied by a frictional resistance between the particles adhering to them; (3) In cases of repeated loading, after the load had been taken off, the rods would not always return to their original position. Comparing the results of beams with all the bars bent up, and those of beams with all straight bars, there was no material difference in the bond-friction-resistance when the stresses on the cross-section under the load were considered for both cases. The Author believes that, on the one hand, there is a gain due to the rods reaching into and being acted on by the compressive stresses in the beam, and, on the other hand, a loss due to the reduction of the length of the arm of the couple, the loss and gain being about equal. The results of tests on stirrups are discussed at length. The Paper is illustrated by curves showing the results of experiments.

R. S. B.

Reinforced-Concrete Construction in Denmark. E. MOGENSEN.

(Beton und Eisen, Berlin, 1910, vol. ix, pp. 361-4.)

Great progress has been made in concrete construction since 1891 when this mode of building was first introduced into Denmark. This may be due partly to the fact that there are ample supplies of good gravel and stone and that native Portland cement is available. Particulars are given of three circular sand-tanks 30.9 feet in diameter and 16.3 feet in depth. A conveyer passes through the bottom of the tanks occupying a height of 2.62 feet. Above the space is a wooden bottom 4 inches in thickness, so that the available depth for sand is 13.4 feet. The upright sides of the tanks are

7·8 inches thick at the bottom, with an external batter, so that the thickness at the top is 3·15 inches. The reinforcement consists of steel rods 0·31 to 0·62 inch in diameter embedded horizontally and vertically in the centre of the concrete. The horizontal rings are of stouter bars at the base and decrease in size upwards; over the openings are channel-irons. An account is given of three rectangular pits for lime also formed of reinforced concrete, the size of each is 62·3 feet long by 13·12 feet in width and 13·12 feet high, with slab-walls 3·15 inches thick at the top and 6·29 inches at the bottom having external buttresses spaced 6·13 feet from centre to centre. The floor is formed of reinforced slabs 1·96 inch in thickness. The foundations are spread to serve as anchorings for the buttresses to the longitudinal walls. A large concrete vat for beer is described by reference to diagrams; in this case all the angles were rounded and the floor was sloped so as to drain to one edge. Details are given of a water cistern in concrete suspended round a chimney in the form of a ring. The concrete work is supported on pillars, the bottoms of which are inserted into the chimney-wall and bound to it by two rings. The whole of the new work is separated from the chimney by a layer of asphalted felt. A projecting bay of special construction is also described and illustrated. For all these works the calculations have been made by Professor A. Ostenfeld of Copenhagen.

G. R. R.

*Ventilation and Heating of Public Halls.*¹ M. HOTTINGER.

(Schweizerische Bauzeitung, Zürich, 1910, vol. 56, pp. 320-2, 337-40, 349-56.)

The Author was responsible to Messrs. Sulzer Brothers for the design and execution of the ventilating and heating apparatus at the new Casino at Bern. Defining a public hall as a meeting-room which will seat more than 200 persons, he considers that the internal temperature should be kept between the limits of 59° F. and 73° F., and that the relative humidity should not exceed 30 per cent. at 73° F., and 60 per cent. at 59° F. He then discusses the quantity of heat and moisture given off by adults and by gas and electric lighting. If the incoming air has too high a relative humidity the audience will suffer from headache. Incoming air should not have a velocity exceeding 1·64 foot per second. He next gives diagrams of possible positions of inlets and outlets for air in a hall, and discusses their respective advantages. As an example of a difficult problem successfully solved he cites the installation at the new Casino in Bern, and gives many detailed illustrations. The architects have successfully masked all the air-ducts. The generating-plant consists of three Sulzer tubular boilers producing steam at a pressure of 1·5 lb. per square inch, which supply heating-batteries

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxix, p. 408.

placed in special chambers in the basement, fresh air being delivered by fans into these chambers, and led by ducts to all parts of the building. There is also direct radiation provided in the halls quite independent of the warm-air ventilation. In the regulation-chamber in the basement is fixed a special thermometer with a switch having numerous contacts so arranged that the temperature of any room can be read, and the admission-valves in the air-ducts can be regulated from the same point. The air of the large hall is changed 3.4 times per hour, and of the small hall 3.8 times per hour, which is equal to 700 cubic feet per person per hour. The total cost, exclusive of builders' work, was £2,980.

E. R. D.

Timber-Preservation. W. BUEHLER.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1359-68.)

The methods for preserving timber from decay may be roughly classed under two headings, i.e., Burnettizing and Creosoting. In the first process, as now practised, the timber is placed in a closed cylinder or vacuum-retort, and then liquid is injected into the wood under pressure. Zinc solutions are generally employed as a preservative. The use of zinc chloride is not advocated for the treatment of bridge timbers, but sleepers may be so treated. If the solution is too strong it renders the timber brittle; 2 to 5 per cent. is considered the best. Owing to the increased cost of creosote oil in the United States, processes have been developed lately in which small quantities of oil are used. These processes are known as empty-cell processes, as the cell structures are merely coated and not filled. The advocates of the full-cell process claim that it is necessary to fill completely the wood-cells to ensure preservation. For the protection of timber against marine borers in southern waters experience indicates the use of at least 18 to 20 lbs. of creosote oil per cubic foot, and 10 to 16 lbs. in northern waters. For paving blocks it is necessary to apply special oil, so as to prevent expansion by the intrusion of moisture. The Author also refers to combinations of these two processes in which a mixture of zinc chloride and creosote is used, such as the Rutgers, Allerdycce and Card processes.

R. S. B.

Fatigue of Materials. CH. FRÉMONT.

(Le Génie Civil, Paris, 1910, vol. lvii, p. 460 to vol. lviii, p. 76.)

No less than 3,000 draw-hooks are broken annually in France, although of similar section to those which on other vehicles have lasted almost indefinitely. The common explanation of these and

similar breakages is that they are due to fatigue, and it would therefore seem necessary to find some method of testing which would show whether a sample was likely to suffer from fatigue, the susceptibility to which must be regarded as a serious defect. Most of these breakages would appear on close examination to be the immediate result of flaws produced either in the ingot or by subsequently working the material at an unsuitable temperature, but want of homogeneity is at any rate a predisposing cause, and its occurrence is much more frequent than is generally recognized. Cases are quoted to show that the yield of phosphorus in a steel sample may be in some places five times as much as in others. The Author considers the notched-bar impact-test of extreme value, but in carrying it out he would previously examine carefully the sample, and would make the test wherever an ocular inspection suggested a defect, rather than at the point which happened to form the centre of the sample, and might or might not be defective. He would also strongly contest the statement embodied in the Report of the International Association on Test Methods, that the velocity of the blow is immaterial. In considering the question of rivets and welds it is pointed out that the adhesion between the surfaces held together by rivets should on no account be ignored; that such adhesion when nickel-steel rivets are used is extremely low and thus discounts the increased strength of such metal. An apparatus for testing adhesion is described, and attention is drawn to the great difference in this particular due to the method of riveting. With regard to welds, the futility of the ordinary proof-test is insisted on, and a case is quoted where a crane-chain, after having been proved to 15 tons, failed 8 days later with a load of less than half. For testing wire an attempt has been made to secure the equivalent of the notched-bar test by supporting the sample in iron sleeves throughout its length with the exception of the point to be tested, and photographs are given showing the results obtained by such test.

I. C. B.

Air-Resistance of Rectangular Planes. G. EIFFEL.

(Comptes Rendus de l'Académie des Sciences, Paris, 1910, vol. cli, pp. 979-81.)

The apparatus for these experiments consisted of a 70-HP. induction fan, producing in an experimental chamber a moving air-column 1.5 metre in diameter, and having a speed of 10 to 20 metres per second. The experimental planes could be placed in this stream at various angles, and by means of a special balance the total pressure could be ascertained as well as its direction and point of application. Pressure-measurements were also taken at various points of the plane. The ratio $\frac{K_i}{K_{90}}$ was obtained, where K_i is the unit pressure when the angle of inclination is i° . Rectangular planes of various

dimensions were tried, the results being plotted on an inclination base. With a square plane the unexpected result was obtained that the unit pressure rose at about 35° to nearly 50 per cent. above the pressure at 90°. After 35° the pressure dropped very rapidly, and at 50° it had nearly reached the pressure at 90°. Similar results were obtained with the other planes, but not in so marked a degree. Up to an inclination of 10° the experimental results are expressed by the formula—

$$\frac{K_i}{K_{90}} = \left(3 \cdot 2 + \frac{n}{2} \right) \frac{i}{100},$$

where n is the ratio of the side of the rectangle perpendicular to the direction of the wind to the other side.

H. R. S.

Hydrography as an Aid in Irrigation. J. C. STEVENS.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1320-47.)

In the various projects of the United States Reclamation Service, accurate determination of the flow through irrigation canals was required, and in this Paper the methods employed are described. Gauging-stations were established at convenient places, and the head of water flowing over the weirs was periodically recorded, the records being worked up in the central offices. By carrying out the survey in a comprehensive manner much information was acquired which was valuable in laying out extensions of the irrigating systems. It is pointed out that sources of error, which in ordinary river-work could be neglected, must be guarded against in the estimation of the flow through irrigation-canals. The current-meter was largely used in addition to the weirs. Velocity observations were taken at two-tenths and eight-tenths of the depth, and the average velocity thus obtained was taken as the mean velocity in the vertical. This method has proved to be the most reliable. The Author describes the gauges used, and the numerous results, in the form of Tables and curves, are discussed. The work done was chiefly in connection with the Sunnyside Project, in which it is estimated that a flow of 1 cubic foot per second of water applied to the lands "creates for them a valuation of" £16,500. In conclusion, the Author states that no limit of accuracy should be recognized for hydrographic work on an irrigation system.

R. S. B.

Stopping Leakage in a Canal-Bank. GALLIOT.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlviii, pp. 112-22.)

Until 2 years ago, when the new system was introduced, leakages other than those very near the water-line were, on the Bourgogne Canal, usually dealt with by running off the water and, after removing the surface of the inner slope for some considerable depth, depositing a layer of either concrete or puddle, and subsequently replacing the excavations so as to form a protecting covering to the newly deposited layer. This system, besides necessitating a stoppage of traffic, was exceedingly costly, particularly from the fact that as there was nothing to indicate when the leak was stopped, a larger area than necessary was generally treated. In the new system a row of sheet-piling is driven near the canal side of the hauling-path; the piles are then withdrawn one by one, and the resulting cavity is filled with concrete, the full head of water being retained in the canal throughout the operation. Piles having a uniform thickness of 3·3 inches were used at first, but it was found that notwithstanding the utmost care taken in guiding, they were apt to "run" by reason of their insufficient stiffness, while owing to their parallelism difficulty was experienced in drawing, and the sides of the cavity were disturbed. The piles now used have a thickness of 4·7 inches at the bottom, and 50 per cent. more at a point 8·2 feet higher. Before filling a cavity, care is taken that the adjacent concrete is thoroughly cleaned, and that any debris resulting from the falling-in of the sides is removed. The driving was originally done with a "ringing-engine," but latterly the power had been obtained from a 4-H.P. oil-motor, with the result that it has been possible to reduce the number of each gang from thirteen to seven. In the ground usually encountered, the fact of driving the tapered piles causes such consolidation that the leakage into the cavity is sufficiently small to enable the concrete to be deposited in the dry; but should a length of very loose ground be met with recourse can at once be had to reinforced piles, and as soon as the unsuitable ground is passed the former system can be reverted to with equal ease. The total cost, when the core or slab is 6·56 feet in depth, is given as 10·89s. per lineal yard.

I. C. B.

Djidjelli (Algeria) Breakwater. DAUJON.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlvii, pp. 29-68.)

In 1849 a scheme was proposed for constructing a breakwater extending from the Citadel point at Djidjelli in an easterly direction, thus completing the partial protection afforded by a line of reefs. Twenty years later the work was taken in hand, but after a length

of 492 feet had been carried out unforeseen difficulties presented themselves and further work was abandoned till 1905, when it was decided to construct an additional 984 feet. While these new works were in progress a like further extension was sanctioned, thus providing for a total length of 2,460 feet, which would carry the outer or eastern extremity seaward as far as the lighthouse-rock. As originally designed the new work consisted of a mound of natural stone, on which was bedded, at a level of 19·4 feet below water, a superstructure of set blocks, but difficulties encountered in preparing the beds for these, especially where the natural rock was met with at a high level, coupled with the uncertainty of the yield from the quarry, led to this design being abandoned after a short trial and replaced by one in which the mound is carried, in pell-mell block-work, up to a point 1·6 foot above water-level, where a reinforced crown, 4·6 feet in thickness and 36·1 feet in width, is laid to serve as a platform for the Titan, and subsequently to carry a capping the top of which reaches a level of 19·7 feet above water.

This upper work is moulded in situ and is protected on the seaward side by a double row of blocks, also constructed in the same manner. The standard pell-mell blocks, which are of concrete, have a cube of 27·5 yards each and weigh about 54 tons, but some few near the toe are of slightly smaller size. The Titan employed was of the usual girder type and travelled on two metre-gauge tracks spaced at a distance apart, centre to centre, of 26·6 feet. With the exception of the Titan, the block-loading machine, the quarry-cranes and the locomotives, all the plant was driven electrically, and the opinion is expressed that the exceptions were only due to the desire to utilize existing plant. The estimated cost of the work was £220,800.

I. C. B.

Bar-Harbours on the Coast of Oregon. M. L. TOWER.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1348-58.)

The improvement of the entrances to the harbours of Coos Bay, and Umpqua and Siuslaw Rivers, Oregon, forms the subject of this Paper, particular stress being laid on the action of flood-tidal currents in causing troublesome shoals. That portion of the Oregon coast between Cape Arago on the south and Heceta Head on the north, presents a stretch of sand nearly 50 miles long between which the above-named harbours are situated. Coos Bay and Umpqua River have controlling natural headlands and reefs of sandstone on the south side of the channel to direct the current in the gorge or harbour. Siuslaw, however, has sandy shores on both sides of the entrance. As an example of the difficulty in maintaining a jetty in considerable depths, the 800 feet of the outer end of the Coos Bay jetty were built three times to 24 feet above low water. The

400 BOAT LANDING-PLACE AT KLITMÖLLER, DENMARK. [Abstracts.

original type of jetty consisted of large timber cribs of 12- by 12-inch timbers bolted together. The cribs were placed in line, filled with small rock and sunk, and then rip-rap was dumped along the sides. Owing to the ravages of the teredo, this type of jetty has been abandoned on the Pacific coast. The causes of the sand-movement in these ports are traced. The Paper is illustrated by maps.

R. S. B.

Boat Landing-Place at Klitmøller, Denmark. P. SLETTING.

(Ingeniøren, Copenhagen, 1910, vol. xix, pp. 366-9.)

The Paper describes how an accident was turned to good account in improving a landing-place for fishing-boats on the exposed west coast of Jutland. The sand along the foreshore had for some time been gradually washed away, thereby exposing a layer of large boulders which in heavy weather made landing exceedingly dangerous and led to frequent losses of the boats. Some steps had already been taken in the way of removing a few of the boulders, when one winter's night a British trawler was driven ashore and was eventually abandoned as a wreck. After a little time it was noticed that the coast-line was gradually being affected by the stranded vessel, to the advantage of the landing. This improvement continuing, it was decided to purchase the wreck and to fill her with concrete in order to keep her in her favourable position. This was successfully accomplished at a moderate cost; the boulders were removed from the foreshore where required and used for filling the vessel, the concrete being cast all round them. A chart shows the soundings before and after the wreck, and photographic views illustrate the various stages of the coast-formation and of the work on the vessel. The total cost was about £400, and for this modest sum a good landing-place was obtained which can now be used with safety in all weathers.

L. S. Z.

Test of Aeroplane-Propellers. R. E. DUDLEY and A. STEAD.

(American Machinist, New York, 1910, vol. xxxiii, pp. 777-80.)

Five propellers were used for these experiments, 5 feet in diameter, having pitches of 2, 4, 6, 8 and 10 respectively. They were made of the true screw type, and for this purpose templates were carefully developed. Their manufacture is fully described. They were driven by a calibrated electric motor and the thrust was taken by a collar carrying a ball-thrust-bearing and actuating a lever transmitting the thrust to a platform-scale; thus the thrust could be read directly. The relation between the thrust, speed and horse-

power was obtained, the results being given in graphical form. It was found that:

$$\text{Thrust} = C \left(\frac{\text{R.P.M.}}{100} \right)^{1.911}$$

where

$$C = 0.28 \times \text{Pitch} - 0.315$$

also

$$\text{Horse-power} = c \left(\frac{\text{R.P.M.}}{100} \right)^x$$

where x varied between 2.8 for the 8-pitch and 3.2 for the 2-pitch propeller, and c between 0.0344 and 0.0007 for the same pitches.

H. R. S.

*Gyrostatic Action of Rotary Engines on Aeroplanes.*¹

ALB. PH. KAPTEYN.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 929-34.)

These experiments, to demonstrate the gyrostatic action of rotary engines on aeroplanes, were made on small models, an ordinary gyroscope replacing the mass of the engine. They showed that, if an aviator moved his aeroplane suddenly to the right or the left, the latter would tip forward or backward, according to the sense of rotation of the engine, this tendency being specially noticeable with rotary engines of the Gnome type. Applying the results of his experiments to a Bleriot machine of the latest construction, the Author found that, if the machine be turned through 90° in 3 seconds, a couple of 825 lb.-ft. is set up tending to tip the machine forward or backward. He supports his contention by citing numerous recent accidents where aviators have fallen to earth without apparent cause except the taking of a sudden turn, and shows that gusts of side-winds would have the same effect. A graph shows the couple produced when a Bleriot machine passes through a right angle at various speeds. Means for overcoming these effects are described, and may be classed under two headings, viz., the Wright machine, where the two rotating masses revolve in opposite directions but not on the same axis, and the Wills-Cloud machine, where the engine is carefully balanced and the two propellers rotate in opposite directions, while their axes are in the same line. In conclusion, the Author shows that, by using two masses rotating in opposite directions, the gyrostatic effect is entirely removed.

H. C. A. T.

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxix, p. 464.

Clouth Dirigible Balloon.

(Deutsche Zeitschrift für Luftschiffahrt, Berlin, 1910, vol. xiv, Pt. 24, pp. 12-16.)

The aim of the recently-deceased manufacturer, Franz Clouth, as opposed to the general effort to attain high independent speed by constant increase in the size of the balloon, was rather to produce a vessel of the smallest possible size compatible with efficiency. He desired to obtain a balloon which could be handled with the gas-supply available in the case of a Field-transport Division. Its weight, moreover, must be confined to the load capable of being carried on an army baggage-wagon, so as not to prove a hindrance to troops on the march, and it must be adapted to ascend and land on the open plain, with no specially erected shelter, and with no exceptionally trained staff. The dirigible balloon is essentially of the non-rigid type, spindle-shaped in general outline, having a gas-capacity of 65,334 cubic feet, a length of 137·8 feet, and 27·9 feet diameter at the widest part. Full details are given of the method of manufacture as also of the stabilizing and steering arrangements. The mode of slinging the car by wire ropes, and the construction of the car itself are explained by reference to diagrams. The trussed car formed of steel tubing is 22 feet in length and contains the 40-HP. motor, which drives the two double-bladed screws and the fan or blower. The total weight was 3,306 lbs., which left a margin of lifting power of 882 lbs. Some account is given of the different ascents and photographs illustrate the balloon and the car with the machinery. Later a motor of 50 HP. made by the Adler Company was substituted for that first adopted, which, notwithstanding its increased power, was lighter than the one formerly used. Sundry modifications made in the steering-gear are explained in an outline diagram, and a photograph shows the latest appearance of the dirigible. More than 50 ascents have already been made, and 1,240 miles have been covered with a total of 150 passengers, without serious accident. The balloon can be managed by the captain, the pilot and the motor-man, with a staff of 10 men for the ascent and landing, and the motor consumes 26½ lbs. of benzine hourly.

G. R. R.

Wireless Telegraphy on Dirigible Balloons.

(Lumière Electrique, Paris, 1910, vol. 12, pp. 99-104.)

The article first discusses the method of obtaining earth-connection, or its equivalent where this is unobtainable, on a Hertzian wave system, and the methods employed for obtaining the waves. The necessity of reducing sparks to a minimum, in view of the presence of the hydrogen-envelope on a balloon, is shown, the heated currents being kept at least two metres from the hydrogen escaping

from the envelope. Diagrams of the arrangements adopted on the *Colonel-Renard*, the *Gross*, the *Parseval*, the *Republique*, and the use of a kite and the symmetrical arrangement of antennae and counter-weight on long-bodied cars are given. The messages from the *Clement Bayard* are stated to have been heard at 100 kilometres with a power less than 50 watts.

J. A. S.

Improvement in Steering Flying-Machines.

A. and F. G. SMIT.

(De Ingenieur, The Hague, 1910, vol. xxv, pp. 872-3.)

The Authors consider the best relative positions of the carrying, steering and elevating planes of flying-machines. Their experience leads them to adopt the use of V-shaped carrying planes curved slightly in an upward direction, which gives the desired sideways stability. The elevation and increase in speed are obtained by varying the angle between the carrying and tail planes. The side steering is obtained by flexing the end of the carrying planes in such a manner that when travelling through a curve the unsteady swaying effect so often obtained is reduced considerably if not entirely eliminated. The side steering may also be effected by means of relay planes. In this case a force is generated at the end of the plane which may be replaced by a force passing through the centre of forces and a couple. This couple tends to tilt the plane till equilibrium is obtained due to a balancing couple, generated by the difference in the upward thrust under each plane. By this means the steering becomes easier, rapid and more certain; and the air-resistance is much reduced, so that the speed may be considerably increased.

H. C. A. T.

Harlan Flying-Machine. WILHELM KIRCHNER.

(Deutsche Zeitschrift für Luftschiffahrt, Berlin, 1910, vol. xiv, Pt. 20, pp. 30-2.)

It is pointed out as a remarkable fact concerning this novel type of apparatus that Mr. Grulich, the constructor, was able at his first attempt to demonstrate its capacity for flight, although the pilot was a wholly-unskilled aviator. On the second ascent the same pilot travelled 3 miles with eight curves and effected a steady landing. This machine is a monoplane of the dragon-fly type, and is very graceful in form, with a frame constructed of American woods, selected to resist the effects of moisture. As distinguished from the ordinary pattern the body is somewhat fish-shaped in side view. The wood struts are placed parallel to one another on either

side, so as to offer the least resistance to the atmosphere, and are of special oval shape, with the same object. The mode of inserting the wooden rods is stated to embody a new principle, involving the use of oval steel tubes welded to the sockets at the cross ties. Provision is also made for detachable fastenings for the steel wires, which take all the tension and leave the woodwork in compression. The method of carrying the wheels on an underframe, arranged with runners, so that the wheels are capable of sliding right or left, in order to avoid breakage, if the machine on landing is not on a level base, is also new. The axle is fitted with rubber rings by way of springs. The wings, covered with rubber-coated fabric, have a span of 43.45 feet and are each 8.2 feet in width. The slightly curved wing-surface is thus about 323 square feet in area. A material advantage is gained in making a considerable portion of the wings perfectly elastic. In this way the back part of the wing-surface adjusts itself to the speed and loading of the machine so that full use is made of the suction of the air-current, due to the ascensional motion. A simple method has been devised of attaching the wings to the body by means of two triangles formed of steel tubing, and this, together with the fastening of the stabilizing planes, both vertical and horizontal, at the rear, is explained by diagrams. The steering-devices are said to be extremely simple and under perfect control. Motion is imparted by a wooden two-bladed screw 8.52 feet in diameter fixed in the front of the machine and driven by a 55-HP. Argus motor, working at 1,300 revolutions per minute. The position of the aviator in regard to the wings and steering-gear is such as to give him a full view of the direction in which he travels, and if a passenger is carried his place on a lower level is such as not to interfere with the look-out.

G. R. R.

Combustion and Boiler-Efficiency. E. A. UEHLING.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 2023-77.)

It is pointed out that, unlike the steam-engine, permanent adjustments cannot be made in a steam-boiler to obtain the maximum efficiency, and therefore the economical working of a boiler depends much on the skill of the attendant. The two main losses exhibit themselves as sensible heat and potential heat; the first varies directly as the flue-temperature and inversely as the CO_2 . The latter is independent of the temperature but varies inversely as the CO_2 . Hence the flue-gas temperatures alone cannot be taken as an index of losses, but the CO_2 must also be taken into consideration. The amount of oxygen in the flue-gases is also an index of the economy but it cannot be so readily determined as the CO_2 . In an Appendix a classified tabular statement of 1,130 flue-gas analyses is given, arranged in groups of increasing CO_2 . These results are

also plotted, and it is shown that the CO_2 and the O vary inversely. On these diagrams diagonal lines are given showing the theoretical CO_2 and O in each case, and the actual percentages obtained are then easily compared with the theoretical values. The percentage of CO is also plotted, but it does not appear to have any relation to the percentage of CO_2 . The best practical percentage of CO_2 is shown to depend on the class of coal burned, and the matter is discussed in detail.

H. R. S.

Transmission of Heat in Surface-Condensers. G. A. ORROK.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 1773-1833.)

This extensive series of trials to determine the heat-transfer in surface-condensers, under various conditions, was undertaken to assist the design of turbo-condensers in view of the higher vacuum necessary and of the great disparity in the existing data on the subject. Existing data from eighteen different sources, giving the thermal units transmitted per square foot per degree per hour are plotted on a water-velocity base, together with the proposed design-curve obtained from the experiments under review. The apparatus used is fully described, but, as first arranged, the errors were as much as 20 per cent. Some minor alterations in the pipe-work greatly reduced this error, but after much investigation the principal difficulty was found to be due to taking the steam from a large exhaust-pipe, causing somewhat unexplained superheating effects. A small steam-heated boiler was substituted, and the accuracy of the results was then satisfactory. Tests were made with five different vacua, varying between 7 and 27 inches of mercury, the circulating water having a temperature of 40°F . and a velocity of 8.6 feet per second, with sixteen different kinds of tubes, including brass, copper, zinc, steel, glass, etc. Other tests were made to determine the effect of velocity, and the result obtained was that:

$$U = 308 \sqrt{V_w},$$

where U is the B.Th.U. transmitted per square foot per hour per degree difference and V_w is the water-velocity in feet per second. This result is compared graphically with results obtained by other experimenters. These results vary greatly. Mr. Orrok's curve occupies a mid position.

The following expression gives the results of the experiments :

$$U = K \frac{C \rho^5 \mu \sqrt{V_w}}{\theta^{\frac{1}{2}}},$$

where C = the cleanliness coefficient varying between 1.0 and 0.5.
 μ = the material coefficient varying between 1.0 and 0.17.
 ρ = the steam-richness ratio, $\frac{P_s}{P_t}$ varying between 1.0 and 0.
 V_w = the water-velocity in feet per second.
 θ = the mean temperature difference.
 K = a constant, probably about 630.

P_s is the partial pressure due to steam, and P_t is the total pressure due to the steam and the air.

The Paper is accompanied by Tables giving the readings and results of 727 tests.

H. R. S.

Hamburg Vacuum-Apparatus for Disinfection by Formalin.

Dr. R. HANNE.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 925-8.)

The Author has had opportunities of fully testing this plant, which was produced in consequence of the necessity for some efficient method of disinfecting skins, furs and other delicate materials which will not stand exposure to steam at 100° C. The Hamburg apparatus has now been in use for 4½ years. In its present form, as constructed by Mr. R. A. Hartmann of Berlin, it comprises four separate sections: the installation of the steam-boiler; vaporizer for formalin; disinfection-chamber; and steam-jet for spraying, known as the "ejector." Any boiler capable of furnishing steam at 45-lb. to 60-lb. pressure may be employed. An illustration of the machine is given. The vaporizer consists of a small horizontal cylindrical vessel, with a capacity of about 3.53 cubic feet, placed, preferably, above the disinfecting-chamber. This vessel contains a coil of steam-pipe, fed from the boiler, and is furnished with a thermometer and vacuum-gauge. The formalin-vapour produced in the vaporizer is drawn off at the top by means of a pipe of ample size, leading to the disinfection-chamber, which is fitted with a steam-coil for preliminary heating up to a temperature of about 50° C. A manometer is provided, to test the exhaustion, and a thermometer, fixed in the door, with a second thermometer placed near the ejector, serve to determine the temperatures. The ejector, situated outside at the base of the chamber near the floor-level, is a simple steam-jet, supplied from the boiler. The mode of working is explained, and the results of practical tests with spores of potato-bacillus on two different dates are given. These tests are compared with those obtained in another well-known disinfecting-machine, that of Rubner, and it is pointed out that a simple steam-jet is here substituted for Rubner's rather complicated air-pump; that the Hamburg

apparatus works with a less amount of vacuum and a higher temperature than that employed by Rubner, and that in this case only a 2-per-cent. formalin solution is needed, in lieu of Rubner's 8-per-cent. solution. The working is simpler and cheaper, and the results are quite as good.

G. R. R.

First Large Gas-Engine Installation in American Steelworks.

E. P. COLEMAN.

(Journal of the American Society of Mechanical Engineers, New York, 1910,
vol. xxxii, pp. 1837-1928.)

The installation of sixteen 2,000-I.H.P. gas-engines working at the Lackawanna Steelworks is fully described and illustrated in detail. Blast-furnace gas is used and the greater proportion of the power obtained is employed for blowing the furnaces. There are also eight 1,000-I.H.P. gas-engines direct-connected to 500-kilowatt generators. Some idea of the size of the installation can be gathered from the fact that the gas passes from the dust-catchers through a 96-inch pipe to a set of four cooling-towers 12 feet in diameter and 72 feet high. Dimensioned drawings are given of the spray-nozzle used in these cooling-towers and of the gas-engines, which are of the Koerting type and were built by the De La Vergne Machine Company. It is stated that the cylinder relief-valves require cleaning once a month and the jackets and heads once a year. Observations have been taken as to the wear of the various parts and their life estimated: thus, the pistons have a life of 3 to 5 years; the piston-rings 2 years; inlet-cams 6 years. The total operating-costs amount to 41s. per annum per horse-power. The results of numerous trials are given, mostly in graphic form; for instance, the thermal units required per blast-horse-power per hour are plotted on a mean-pressure base, and it appears that the most economical mean-pressure is about 49 lbs. per square inch. Down to 43 lbs. per square inch the economy is not greatly reduced, but for smaller mean-pressures the loss of economy is rapid. Various losses, such as pumping, friction-losses, incomplete combustion, jacket- and exhaust-loss are also plotted on a mean-pressure base. In many cases the observation results are somewhat erratic. There are numerous appendixes, giving information as to the best point of ignition relatively to the speed of rotation, diagrams showing losses due to the air- and gas-pumps, and a complete set of the results obtained at the trials, accompanied by numerous indicator-diagrams. The gas was measured by means of a Venturi meter.

H. R. S.

Gas-Producers.

(Revue de Mécanique, Paris, 1910, vol. xxvii, pp. 555-94.)

This article contains descriptions and sectional drawings of a large number of recent suction-gas producers. It is observed that none of them fully conform to the conditions of perfection laid down in an article in the *Revue de Mécanique*, December, 1907, and that the difficulties caused by clinkers and tar have in a great measure still to be overcome.

The Akerlung producer, intended for using bituminous coal, is of the inverted type, i.e., the combustion takes place at the top of the producer and the gases are withdrawn from the bottom, the object being to gasify the tars. Theoretically the arrangement is perfect, but difficulties arise in dealing with the ashes. The Bollinckx producer has some special points; the gases are collected by means of an inverted cast-iron cone placed some distance above the furnace. The latter has no fire-brick lining to protect the cast-iron shell, which is kept cool by means of water trickling on the outside, the water-vapour thus produced being drawn into the producer. The usual fire-bars are replaced by a flat water-cooled plate, above which there is a plate with a hole in the centre, which can be rotated by means of gearing. The rotation of this plate causes the ashes to fall out, and they immediately drop into water. It is found that the clinkers do not stick, and there is the further advantage that the vaporizer acts instantaneously on change of load. Several two-zone producers are described, in the first of which the hydro-carbons are distilled, and in the second zone these distillates and the coke produced are gasified. The Fielding and Platt, the Jabs and the Westinghouse producers come under this category. In respect to the latter the results of experiments are given in graphical form, and it is shown that at the full rating of the engine 18,000 cubic feet of gas per hour were produced, but that when the engine was running idle it was necessary to make 6,000 cubic feet of gas, and since the efficiency of the producer had dropped from 78 per cent. at full load to 66 per cent. at no load, it follows that the coal-consumption at no load was four-tenths that at full load. A large number of producers with rotating and oscillating grates are described, also many arrangements for distributing the charge and for poking the fuel.

H. R. S.

Heavy-Duty Gas-Producers. N. LATTA.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 2073-88.)

Gas-producers were originally a failure in the United States, owing to the acceptance of the English data of burning 20 lbs. of coal per square foot of grate. It has been found that, with

American coals, this rate is excessive, due to the amount and quality of the ash which readily fuses into clinker, and that 8 to 9 lbs. per square foot of grate is the maximum possible. It appears that 750 HP. is the largest size which has been developed with a reasonable degree of commercial success.

In the producer here described the new principle has been adopted of pulverizing the fuel and blowing it in with the requisite amount of air into the lower part of the producer, and by this means sufficient gas for 3,000 HP. has been obtained from a producer 19 feet high and 8 feet 6 inches in diameter. It consists of two concentric rings of fire-brick, 9 inches thick, leaving an annulus 18 inches wide; the lower part of the annulus forms the combustion-chamber and the upper part the gasifying-chamber, the annulus being constricted between these two chambers. The upper part of the gasifying-chamber is in connection with the central hollow cylinder of brickwork, which forms the down-comer for the producer-gases. A sectional drawing of the producer is given, together with several photographs and an analysis of the producer-gas obtained.

H. R. S.

Fine Grinding in Tube-Mills. H. W. HARDINGE.

(Engineering and Mining Journal, New York, 1910, vol. xc, pp. 1057-8.)

The Author directs attention to the unsatisfactory conditions under which tube- or pebble-mills are at present worked. His investigation of the problem of fine grinding in this class of machine has led him to believe that the work of division is done much more by percussion than by compression or attrition. Hence it follows, from a consideration of the complex action going on in a tube-mill, that the error in practice which occasions so great a loss of efficiency lies in not suitably grading the materials fed into the mill. Engineers use a large crusher to break an 8-inch block of ore, and a smaller one to deal with a 4-inch mass. But in the case of a tube-mill for fine crushing, they use the same machine to break a 20-mesh particle down to a 200-mesh, thereby reducing the crushing-efficiency of the mill to only a little more than that of a wagon-wheel rolling over a sandy road. The comminution being effected by impact—a shattering of the particle—it is obvious that the cushioning effect of much smaller particles is destructive of efficiency. The remedy seems, therefore, to lie in feeding to the same mill particles less diverse in size, or in some device for removing the “fines” as they are formed. Two photomicrographs of quartz ground in a tube-mill are given to show that the action is percussive, and that the finished product is still made up of particles of very different sizes.

G. G. A.

Field for Grinding. C. H. NORTON.

(Journal of the American Society of Mechanical Engineers, New York, 1910, vol. xxxii, pp. 2113-26.)

The Author contends that present-day grinding cuts the metal just as if the wheel were a very fine milling-cutter, and does not remove it by abrasion. Microphotographs to prove this contention are given. The chips from the modern grinding-wheel have every appearance of lathe-chips, whereas those obtained with the old-form emery-wheel are rounded and fused, indicating the high temperature reached with the old method of grinding. It is stated that, in some cases, with the new form of grinding-wheel there are as many as 2,000,000,000 cutting-points per minute, 400,000,000 per minute being quite common. Numerous examples of work done by grinding from the rough, without any lathe work, such as cams, crank-shafts, lathe-spindles, connecting-rods, etc., are given, together with the amount of material removed, the time required for the job and the limits of accuracy attained.

H. R. S.

Design of Hoisting-Hooks. A. K. PEDERSEN.

(American Machinist, New York, 1910, vol. xxxiii, pp. 954-5.)

The Author has prepared a chart (published drawn to a large scale and with full explanations as to its use) to facilitate the calculation of the dimensions of hoisting-hooks. Three curves are given on the chart: one based on Professor Pearson's theory of curved beams, the second on Professor Bach's theory, the variation between these two curves being of the order of 4 per cent. The third curve is based on the straight beam theory, and hooks designed by means of this curve would be stressed 50 per cent. more than those designed in accordance with the two previous curves. It is marked "Not for use," as it has been shown to be unreliable by recent experiments. In using this chart a factor has first to be calculated from the formula—

$$F = \frac{S}{P} \cdot A^2 \tan \frac{T}{2},$$

where S is the maximum allowable tensile-stress, P is the load on the hook, A is the radius of the inside of the hook, and T is the angle of taper of the horizontal section of the hook. The ratio between A and the depth of the hook (at the horizontal section) can then be obtained and the remaining dimensions follow.

H. R. S.

Electric Lifting-Apparatus. DEHENNE.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. ii, pp. 333-65.)

The first patent for lifting-apparatus with electric control was taken out in France in 1878, and in the same year was effected the first application of electric transmission of energy to a lifting-machine. The machines under consideration may be divided into three classes, according as the motion produced is in one, two or three directions. The element common to all three classes is the winch, and descriptions of electric winches, endless screws and electric limiting apparatus are given. Working appliances may be classed as effecting combining and effecting contact; the controller used on electric trams is the type of the former class, and of the latter class, which are used to establish continuity of the circuits, those worked by hand and by magnetic relays are described. In the first class of machines are described some Parisian lifts, extracting-machines for mines and hoists; in the second class cranes and harbour-cranes; and in the third class 5-ton, 10-ton and 15-ton travelling-cranes, details of construction and data of speed and efficiency being given. The Author considers that the speed of working must be increased, metallic cables used, that the use of a single motor must be abandoned in favour of the adoption of at least one for each movement, and that the angular velocity of the electro-motors must be reduced.

A bibliographical appendix follows the Paper.

W. H. B.

Experiments with Oil-Separators.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 1969-75, 2018-26 and 2104-7.)

Sixteen different types of oil-separators were tested at the Munich Testing-Station of the Bavarian Revisions-Verein to determine the influence of construction, the quality of the oil, and the temperature of the steam on their efficiency. Records were taken of—(1) Steam-consumption per hour; (2) Steam-pressure and temperature before and after the cylinder; (3) Quantity of oil supplied to the cylinder; (4) Drop in pressure of the steam passing through the separator; (5) Oil-contents of the steam before and after the separator.

The engine was run both condensing and non-condensing, and on superheated and saturated steam. The results may be briefly summarized as follows:—Separation down to 0.022 or 0.033 lb. of oil per ton of water may be considered sufficient, if it is merely a question of cleanliness of the condenser-surfaces and of economy in oil-consumption; should the condensed water be used again as boiler feed-water the separation must be completed in suitable

filters. Centrifugal separators were found to be the best, and especially such in which the oil was immediately removed from the presence of the steam as soon as it was separated. Those separators which could be adjusted for varying quantities of steam showed a considerable advantage over the other types. It was found that the temperature and the flash-point of the oils used were the determining factors in separation, fatty oils being the most difficult to separate and condense. The temperature of superheated steam should not exceed 572° F., as above this large quantities of lubricating oil would be evaporated and brought over with the steam. It is, therefore, advisable to admit the oil at the coolest part of the cylinder, and not with the steam, as is the general practice at present.

C. J. G.

Making Patterns from Oiled Sand.

(American Machinist, New York, 1910, vol. xxxiii, pp. 1045-9.)

A detailed description is given of a novel method of making patterns with oiled sand. Moulding-sand is well oiled so that each grain is coated; the mass is approximately shaped to the pattern required; it is then baked, when it becomes hard, and can be cut and sawn quite readily. Prints can be put on, and, in fact, it can be treated almost as if it were a piece of wood. When the pattern is completed it is rebaked, then rubbed down and varnished. It is stated that such patterns have been used 100 times and then appear as good as new. If broken or chipped they can easily be repaired. After each repair the pattern must be rebaked.

H. R. S.

Effect of Key-Ways on Shaft-Strengths. H. F. MOORE.

(University of Illinois Bulletin No. 42, 1909, and American Machinist, New York, 1910, vol. xxxiii, pp. 1200-3.)

An account is given of a series of experiments made at the University of Illinois to determine the reduction in strength of a shaft caused by key-ways when exposed to pure torsion and to combined torsion and bending. Three different proportions of key-ways were tested of the ordinary type, as well as a Woodruff key which is circular in the direction of the axis of the shaft. Cold-rolled steel shafting was used, of 1½ inch to 2½ inches diameter, and the tests consisted in determining the point at which yield took place, both in the plain shaft and in the shaft fitted with the key-way. The ratio between these two loads the Author calls the "efficiency of the shaft with key-way," and the results of the experiments show that the efficiency is

$$e = 1.0 - 0.2w - 1.1h,$$

where w is the width of the key-way divided by the diameter of the shaft, and h is the depth of the key-way divided by the diameter of the shaft. Incidentally the following values were found for cold-rolled steel shafting:—the modulus of torsional elasticity 12,000,000 lbs. per square inch, and the elastic limit in torsion 39,000 lbs. per square inch.

H. R. S.

Experiments on Ball- and Roller-Bearings. CH. GÉGAUFF.

(Le Génie Civil, Paris, 1910, vol. lviii, pp. 93-6.)

The apparatus used in the experiments embraced a horizontal central shaft which carried the inner ball-race, and could be rotated mechanically at different speeds in either direction, and an outer ball-race carrying a drum, over which a flat steel band was hung in such a manner that the depending ends could be loaded either directly or through the intervention of levers so as to produce the pressure on the bearing required by any particular experiment.

The friction was measured by noting the excess load, which it was necessary to place on one end of the band in order to restore equilibrium when the central shaft was rotated.

Four systems of bearings, classed according as the points of contact, are: (a) Two; (b) Three; (c) Four, which when joined together form a rectangle; and (d) Four which, when similarly joined do not form a rectangle, are discussed. The conclusions are that preference must be given to the various systems in the order named, and that (d) may absorb as much power as a plain bearing, i.e., about thirty times that absorbed in (a). The diameter of the ball is of great moment, and the natural conclusion that the larger the ball the less the friction is entirely wrong. The best diameter of ball was found to be 0.08 inch plus one-seventh the diameter of the inner rolling circle. The question of high speeds is mentioned, and it is stated that for spindles revolving very quickly, such as those of spinning-frames, the ball-bearing has no advantage owing to the effect of centrifugal force. A Table is appended showing the maximum permissible loads which ball-bearings of various diameters, supporting shafts running at different velocities, will carry. When heavy loads are to be dealt with a multiplication of the number of ball-rings is recommended in preference to any departure from the proportioning of the bearing. Rollers are only recommended where the paths cannot be hardened, and it is stated that the friction in a roller-bearing never falls below four times that in a well-designed ball-bearing, while it may increase almost to any extent through want of parallelism between the rollers and the shaft.

I. C. B.

Ball-Bearings for Railway Coaches and Wagons. A. BAUM.

(Organ für die Fortschritte des Eisenbahnwesens, Wiesbaden, 1910, vol. xlvii, pp. 375-8 and 394-6.)

The wear on ordinary fixed journals amounts to 0.176 lb. for passenger-coaches and 0.33 lb. for goods-wagons per axle per annum. This wear amounts to about 340,000 lbs. of white metal, costing £15,000 per year on the Prussian Hessian State Railways alone. The amount of oil consumed is 2,610 tons, costing £35,000 a year, and in addition there are annual totals of £50,000 for wages and £20,000 for renewals.

The ball-bearing here described consists of two concentric ball-bearing rings provided with three accurately-turned ball-races. Should a ball in one of the races become broken the pressure is automatically redistributed over the other two races, while with three races smaller balls can be used. These ball-bearings were tested for 3 years, the trucks to which they were fitted being used for rough work in connection with steel-works. After this test not a single ball was found to have been broken. The cost of these bearings is 49s. each, as against 12s. 8d. for the old type of bearing. Exhaustive tests showed, however, that the reduced oil-consumption, wear and renewals more than balanced the increased initial expenditure. Six trucks were next compared, three of them being fitted with ordinary and the other three with ball-bearings. The reduction in the tractive effort required was 50 per cent. for empty and 70 per cent. for loaded trucks. The oil used was reduced by 75 per cent., equivalent to 2,087 tons per annum. At higher speeds the saving in tractive effort would not be so great, since the tractive effort T is given by the formula $a + bv^2 + \frac{1,000}{n}$, and the use of ball-bearings only affects the term a in the equation, which would be approximately halved. The Author concludes that ball-bearings can be used most advantageously for goods- and suburban trains where frequent stops and medium speeds are the rule.

C. J. G.

Distribution in Steam-Engines with Separate Distributors.

LETOMBE.

(Mémoires et Travaux de la Société des Ingénieurs Civils de France, Paris, 1910, vol. ii, pp. 366-75.)

The Author's aim is to replace the approximation in general use by an exact diagram of the motions to be realized, apart from any particular ideas adopted for the entrance and outlet of the steam. When once the details of the distributors, gear and eccentrics have been decided upon, it remains only to find by a diagram the lengths of the levers and the coupling-rods, and to arrange between them

the various parts so as to realize the proposed phases of distribution. Taking, as the case to consider the Corliss engine, the Author investigates first of all the problem: given the two angles described by levers, set in any fashion whatever in a plane, about two fixed points, to determine the lengths of the levers and connecting-rod, so that an angular movement of one of the levers corresponds exactly to that provided for in the other. In this case, no limit is set to the angle of lap, and the Author then proceeds to study the problem on the supposition that this angle has been fixed in advance. Finally is considered the case in which it is desired to reduce the angle of lap as much as possible. Each case is accompanied by a diagram, and the solution is applicable not only to steam-engines but to any machine with articulated quadrilaterals.

W. H. B.

Steam-Turbines. J. KERSTEN.

(Revue Universelle des Mines, Liège, 1910, vol. xxxii, pp. 1-58.)

Leaving theoretical considerations aside, an obvious feature in the turbine of to-day is its organic superiority over the piston-engine. The only parts subjected to friction are the shaft-bearings, which may be lubricated to reduce the resistance to a minimum. The best form of blade being ascertained, the wearing action of the steam on it is of little importance, so that the organic efficiency is not appreciably diminished by wear and tear. In a piston-engine working under a full load the organic loss is usually about 13 per cent. when new, and often twice that amount when old and worn. It has been ascertained by experiment with a Parsons turbine driving a dynamo of 1,000 kilowatts, that the coefficient of total loss is barely 11 per cent. under a full load, while an instance is cited where no appreciable wear was discernible after a 2-years uninterrupted run. Moreover the thermic cycle is continuous in a turbine, the steam always flowing in one direction, so that any given portion of the surface is constantly maintained at the same temperature. Thus cooling and condensation are avoided; and further, expansion may be more fully utilized. It follows that the thermic efficiency of the turbine is superior to that of the piston-engine. A valuable property of the turbine, especially for use in power-centres, is the fact that the steam-consumption per unit of power increases but slowly as the load diminishes. The Author mentions the case of a turbine which consumed $14\frac{3}{4}$ lbs. of steam with full load, $16\frac{1}{4}$ lbs. with a half-load, and $17\frac{1}{2}$ lbs. with a quarter-load per kilowatt-hour. A saving of oil, which may be as little as one-eighth of that required by a piston-engine of equal power, is an advantage of some practical importance. Occupying only about one-third of the space required for a piston-engine very powerful units may be constructed, up to at least 20,000 HP. A disadvantage possessed by the turbine is its necessarily high speed,

which for the present limits its use mainly to electric power-centres and to ship-propulsion. The steam-consumption of the turbine depends largely on the power developed, the degree of superheating and the vacuum. For a Brown-Boveri-Parsons turbine of about 750 kilowatts it has been found that the consumption diminished 1 per cent. for an increase of 1 atmosphere up to 11 atmospheres. For larger machines the diminution was $1\frac{1}{2}$ to 2 per cent. Above 11 atmospheres the influence appeared to be nil. Superheating has a more marked effect. As a general statement it may be affirmed that the consumption decreases 1 per cent. for an increase of 6 to 7° C. in temperature. The vacuum also plays an important part in the economical working of a turbine, for the latter allows of a much more complete expansion of the steam, which may be carried to as much as 140 times its original volume. Experience here shows that the consumption varies 1 per cent. for $1\frac{1}{2}$ to 1 per cent. of vacuum. The Author states that the steam-consumption of a turbine is not less than that of a piston-engine of equal power till a power of about 500 kilowatts is reached. Beyond that the advantage rests with the turbine. Various types of turbine are described and illustrated.

G. G. A.

Constant-Revolution Water-Turbines working under Variable Head. B. BACHMETEFF.

(Zeitschrift für das Gesamte Turbinenwesen, Berlin, 1911, vol. viii, pp. 7-9.)

In water-turbines it is important to know the power developed at constant revolutions when working with variable head. The information can be deduced from a series of brake-trials under a constant head with various powers and variable speed. Such data are given in the article and on plotting these figures it is found that the desired relation follows a straight-line law. This law is compared with the results obtained from several other turbines and is confirmed. A simple rule is thus deduced. The turbine under consideration is run approximately with the maximum head, giving the maximum load and one point on the straight line; it is then run without any load, giving a second point; the straight line drawn through these two points is the relation desired.

H. R. S.

Tests of a 200-Kilowatt Turbine.

(Zeitschrift für das Gesamte Turbinenwesen, Berlin, 1910, vol. vii, pp. 481-4, 489-503, 516-18; and 1911, vol. viii, pp. 23-7, 71-4.)

These tests of a two-wheel Curtis turbine, manufactured by the Allgemeine Electricitäts-Gesellschaft, are of a high order of accuracy. The following are some of the points investigated:—The dynamo-

and friction-losses of the turbine were determined by running the generator as a motor, and it was found to be essential to have an absolutely-constant exciting-current, which was obtained from an accumulator-battery. The iron-losses in the generator were ascertained at speeds of 2,000 to 3,000 revolutions per minute and with varying excitation. At the former speed, with 6 amperes exciting current, the loss was 0.7 kilowatt, and with the latter speed 2.6 kilowatts; the law of loss was parabolic. The steam-friction loss in the turbine was found to follow a linear law when plotted on logarithmic co-ordinates, and was also found to vary as the 2.76 power of the speed, which is a smaller index than that given by Stodola, namely, 3. The constant given in Stodola's steam-friction formula for rotating disks was found to be 4.42, which substantially agrees with Stodola's value. The heat-radiation loss for each stage of the turbine was also determined and was found to be equal to 41.9 calories per hour per degree Centigrade for the first stage, and 26.4 calories per degree Centigrade for the second stage. The total loss by radiation was found to be 4.75 per cent. of the maximum power. The results of the research are tabulated in great detail and are also shown in graphical form.

H. R. S.

Increased-Capacity Turbines at Niagara Falls, Ontario.

(Electrical World, New York, 1910, vol. 56, pp. 1525-7.)

The old and new types of turbines at Niagara Falls are illustrated and contrasted. The new turbine was designed by Mr. C. C. Egbert, the mechanical engineer of the Niagara Falls Power Company, to deliver 12,500 HP. at reasonably good efficiency, and drives a three-phase 25-cycle 12,000-volt generator. The turbine is of the inward-discharge, Francis type with draft tubes led to the bottom of the wheel-pit. A regulation-gate at the end of the wheel-pit maintains the water at a sufficient level to cover the mouths of the draft tubes at all loads. The weight of the rotor is carried by an oil-thrust-bearing under the generators and a hydraulic piston at the bottom of the wheel. The governor is of a modified Escher-Wyss design. Drawings of the cross-section of the new turbine, a cross-sectional elevation of the old and new units, and a plan-view of the new unit are given. The difference of design in the old and new turbines is principally one of detail for the purpose of obtaining 25-per-cent. greater capacity on the new unit. When operating under 133 feet head the set gives 12,800 HP. at the switchboard. The efficiency is apparently equal to or greater than that of the older turbines, and the cost of construction is in favour of the new design.

J. A. S.

Tests of Traction-Engines at Winnipeg, Canada.

(Engineering News, New York, 1910, vol. 64, pp. 441.)

The classes tested were: (a) Internal-combustion machines of 20 B.H.P. and under; (b) Internal-combustion machines of 21 to 30 B.H.P.; and (c) Above the latter power: and the basis of awards was brake-test, hauling, ploughing, and design and construction, the maxima being 150, 100, 200 and 50 respectively, total 500. The Table which forms the bulk of the article, in addition to the description of the tests, gives every particular of the machines. Steam-machines were also included.

C. O. B.

Further Researches in Coal-Dust Explosions.

(Bulletin de la Société de l'Industrie Minérale, Saint-Etienne, 1910, vol. xiii, pp. 319-403.)

This is a detailed and critical report of recent researches and experiments made at the Testing-station at Liévin¹ for the purpose of ascertaining the conditions under which a coal-dust explosion may originate. The inquiry in this series of experiments was limited to the relative proportions of dust and air in a possible mixture, the degree of fineness of the dust, the percentage of volatile matters in the dust, the effect of mixing stone-dust with coal-dust, and the degree of watering necessary to render the dust unflammable. These investigations were carried out under conditions existing in a mine with a view to arriving at some reliable practical results, especially with regard to the proposed stone-dust remedy. The conclusions reached, in so far as the inquiry was concerned with the originating of an explosion, are set forth as follows:—The risk of an explosion is very small and may for practical purposes be accounted nil—(1) When the quantity of coal-dust existing in a free state is less than 112 grammes to the cubic metre of air with which it may become mixed; (2) When the dust is in the form of granules of 1 to 2 millimetres diameter, with only a small proportion of dust fine enough to pass a 200-mesh sieve; (3) When the coal contains no more than 18 per cent. of volatile matter; (4) When the incombustible elements (stone-dust) constitute at least four-tenths (40 per cent.) of the dust mixture; and (5) When the coal-dust is moistened by at least its own weight of water. These considerations make it possible not only to determine when a mine is safe from a pure dust explosion, i.e., without the intermixture of fire-damp, but also to provide effective means for rendering a dangerous place safe. By preventing, as much as possible, the accumulation of coal-dust, the stone-dust remedy, which promises

¹ See Minutes of Proceedings Inst. C.E., vol. clxxxii, p. 379.

the best results, is made more easily practicable. The Author strongly condemns the practice of erecting screens near the intake ventilating-shafts, and recommends the sprinkling of the "tubs" to lay the dust when taken on to the cage to be lowered.

G. G. A.

Restricting the Area of a Coal-Dust Explosion. J. TAFFENEL.

(Bulletin de la Société de l'Industrie Minérale, Saint-Etienne, 1910, vol. xiii, pp. 553-99.)

The difficulty of preventing a coal-dust explosion, or even of limiting its effects to a small area, by any system of watering surfaces or "schistification" of deposits, i.e., addition of stone-dust, which recent experiments at Liéven¹ have shown to be practically insurmountable, has compelled attention to a different method of applying these remedies. The "stop-barrage," the name suggested for the method, is intended to arrest the further progress of an explosion by extinguishing the flame when it has reached a certain point in the roadways. This was first attempted by heavily watering the ways, or thickly dusting them with stone-dust, at such selected points. But experiments have shown that such means are insufficient to stop an explosion which began with violence due to the presence of fire-damp at the place of ignition. Recourse, therefore, has been had to the "barrage," which these Liéven experiments have shown to be successful under all conditions. The water-barrage consists of small vats or barrels filled with water, and pivoted, or otherwise arranged, to be overturned by the advancing air-wave in front of the burning gases. This air-wave not only tips the receptacles, but scatters and sprays their contents so that the closely following flame is suddenly cooled and extinguished. The stone-dust barrage is still more simple and not less effective. It consists of a few broad planks placed transversely near the roof and covered with a thick layer of stone-dust. This is blown off by the air-wave, thereby creating a cloud of unflammable dust in which the advancing flame is extinguished. Both methods have been proved to be capable of stopping the most violent explosion.

The quantity of water found to be sufficient in extreme cases is 25 gallons per square yard of the sectional area of the roadway, distributed over a length of 10 yards. With a number of small receptacles the charge is more effectively distributed. For the stone-dust barrage 100 gallons per square yard of sectional area may be considered necessary, distributed over a length of 10 to 15 yards of roadway. It should not be in too thick a layer on the cross planks, and the latter should be sufficiently spaced to allow the dust to fall freely between them when blown off by the explosion, say, about 2 feet. Where the roof is high, the planks may be set in two

¹ See preceding Abstract.

stages, one above the other, leaving however a clear space of at least 2 inches between the dust-layer and the roof. The dust must be fine, the largest particles not exceeding $\frac{3}{4}$ inch in diameter.

G. G. A.

*Purchase of Coal by Calorific Unit.*¹ EUG. GRANDMOUGIN.

(Le Génie Civil, Paris, 1911, vol. lviii, pp. 196-9.)

Considering the care with which the essential yield of manures and mineral ores is determined and how the market-price is ruled by such determinations, it would seem somewhat singular that the useful property of coal, viz., its calorific value, is not more employed, if not as a unit for purchasing, at least as a check on the material supplied. Possibly the reason is that, of the factors involved when deciding upon the suitability of any coal for a specific purpose, so many are wholly incapable of accurate valuation that any attempt at so treating the rest would have, on superficial examination, the appearance of an unnecessary refinement. The calorific value of coal may be deduced by Dulong's formula from the elementary composition, but for commercial purposes the method is too complicated, and wherever the unit is employed recourse is had to some form of calorimeter, notably the Berthelot calorimetric bomb. With this instrument the smallness of the sample used makes it necessary to exercise the greatest care, to ensure that this extremely small quantity is a fair representation of the bulk, and a full account is given of how this can best be accomplished. In Germany and Switzerland a correction is made in the value thus determined owing to the fact that in the bomb the water produced by the combustion deposits on the cold walls and thus gives out its latent heat, an action which does not take place in the flue itself. The principle of settling prices on the basis of the calorific value is by no means uncommon in America, while it is constantly employed by the Paris-Lyons-Mediterranean and the Swiss railways. The cost of employing this method is small even if a valuation be made for every 50 tons delivered.

I. C. B.

Safety Electric Motors for Fiery Mines. DR. A. THOMALEN.

(Glückauf, Essen, 1910, No. 48, pp. 1884-6.)

Electric motors used in fiery coal-mines in Germany must be constructed to fulfil the conditions imposed by legal enactments, the main condition being that the motor shall not, through spark-

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxxi, p. 542, and vol. clxxxii, p. 365.

ing, be capable of causing a gas-explosion in the mine. An explosion within the motor is not prohibited, for it is practically impossible wholly to exclude the air; but an ignition inside must not communicate itself to the outside atmosphere. Two methods of meeting this requirement are available. One is to enclose the motor completely in a casing strong enough to withstand an explosion inside it, the gases being retained long enough to cool them. The other method provides for the immediate, and relatively free, escape of the gases through passages constructed to deprive them of much of their heat. These passages are formed by a number of superimposed sheet-metal rings separated one from another by a space of $\frac{1}{2}$ millimetre. Through these spaces the hot gases generated by the explosion pass radially to reach the outer air. The action of this cooling-ring is the same as that of the gauze of a Davy safety-lamp. With such a provision for safety, either the whole of the motor, or only that part from which sparks are emitted, may be enclosed. For ventilated motors the latter arrangement is adopted. In such construction the mine-air has free access to the interior, and only the commutator portion is provided with the cooling-ring device. Since the sufficiency of the latter is of vital importance, it is deemed necessary to subject it to severe tests before sending the motors out from the works. This is done at a specially designed proof-station. An illustration is given of such a station at the Siemens-Schuckert works at Nürnberg. The test, a trying one, is repeated twenty to thirty times with varying proportions in the gas-mixture. Only after passing satisfactorily all the trials is the motor judged to be fit for use. Several photographic views are given of these protected motors.

G. G. A.

Portable Electric Safety-Lamps. P. KLIVER.

(Glückauf, Essen, 1911, vol. xlvii, pp. 57-64.)

German efforts to satisfy the need for a suitable portable electric safety-lamp for rescue work, as well as for examining dangerous places generally, have had considerable success. Eight different types of lamp, with several variations of some of the types, were submitted to the Author to be tested under conditions in which they may have to work. These, now described and illustrated, are all of the accumulator class, i.e., the lighting current is furnished by a lead "secondary" cell previously charged by a direct continuous-current from a dynamo. The principle of these lamps is thus the same in all, their differences being found in construction. The aims of the designers have been chiefly to reduce the weight to a minimum, to obtain the maximum duration of light for a given weight, and to produce a lamp which may be roughly handled, and even dropped, without danger or risk of serious derangement. The Author describes how each of these problems has been dealt with, and in what degree they have been solved, as shown by the results of his tests.

The lamps varied in weight between $2\frac{1}{2}$ lbs. and $5\frac{1}{2}$ lbs.; in duration of light between $4\frac{1}{2}$ and 23 hours; and in first cost between 20s. and 25s. In some of these lamps the light is projected from one side of the containing case with the help of a reflector behind the incandescent filament; in others the bulb is mounted on the top of the case and protected by a glass cylinder and fend-rods. The latter is said to be the better arrangement for a working light, the former for a search light. The design which has been judged the most suitable is the production of the Allgemeine Akkumulatoren-Gesellschaft of Dresden. The electrodes in this lamp, known as No. 2 type, are cast in one piece as a hollow, barred or reticulated cylinder, the former for the positive, the latter for the negative element. The cell, of either vulcanite or celluloid, is securely closed, glass-wool being used for packing and as an absorbent to prevent "creeping" of the acid, or spilling, or any great disturbance even when held upside down. A small glass tube passing through a soft rubber plug affords an outlet for the gases generated. The cover is hinged and locked, the bolt being withdrawable only by means of a magnet. This lamp is recommended for the use of mining officials. Its weight is 4 lbs., and when charged for 7 hours by a current of 0.46 ampere, it will give a light of 23 hours' duration. Its voltage is 2.1 volts at the beginning and 1.7 volt at the end of its discharge.

G. G. A.

Iron-Ore Reserves. PAUL TRASENSTER.

(Revue Universelle des Mines, Liège, 1910, vol. xxxii, pp. 133-82.)

The Author deals very fully with the iron-ore resources of the world as an index to industrial developments in the future, particularly with reference to Great Britain, Germany, France and the United States. He takes account of the absolute known reserves in each of these countries according to the latest estimates, the actual consumption as given by the statistics of 1909, and the probable increase in the latter due to expansion of trade. The quantity of ore mined in Great Britain in 1909 was 15.2 million tons, leaving the known reserves at the estimated value of 1,300 millions. If this rate of extraction be maintained the reserve will last only about 85 years. The quantity mined in Germany in the same year amounted to 25.5 million tons, out of an estimated reserve of 2,810 millions. This rate of extraction, calculated on a similar basis, gives a duration of 110 years. In France the quantity raised was 12.3 million tons out of an estimated reserve of 3,300 millions, a rate of consumption indicating, if maintained, a duration of 270 years. The industrial activities of the United States required an extraction of 57 million tons from a known reserve roughly estimated at 4,800 millions, a rate of consumption which will exhaust these reserves in about 84 years.

The duration of the known reserves, expressed in years, relatively to the production of 1909 is the most significant index of the relative mineral wealth of the countries compared, and, moreover, it shows approximately how long the present rate of production can be maintained without a serious increase in the cost. If account be taken of the growth of the iron industry in recent years, it must be concluded that the United States will be the first to run short of rich ore. For Great Britain and Germany the prospect is equal, for the more rapid expansion of German industries will compel the ironmasters of that country to fall back sooner than their English rivals on their poorer ores. France appears to be in a privileged position, thanks to her resources in the Briey basin.

G. G. A.

Electro-Steel.

(Glückauf, Essen, 1910, No. 47, pp. 1851-2.)

The production of electro-steel is steadily increasing. The number of new works erected during the last year, or now in course of construction, testify to the growing demand for the material consequent on its wider application. Though, from certain causes, the output in Germany fell from 19,539 tons in 1908 to 17,773 tons in 1909, the number of centres of production increased considerably. The world-production in the latter year amounted, in round numbers, to 50,000 tons from 100 furnaces. Progress since made shows that this youngest branch of the steel industry is firmly establishing itself. The electric furnace offers the possibility of producing steel in large quantities equal in quality to that obtained from the crucible and at much less cost. Through this means the manufacturer can now produce rails, axle-shafts, wheel-tyres and girders in metal of the same quality as tool-steel. Several thousand tons of electro-steel rails have already been delivered from German works in Völklingen. These have been subjected to severe tests, and have shown remarkable power of resistance to transverse strain and the shock of blows. With the electro-steel rails of American manufacture these are said to compare very favourably by reason of the superior refining process which they have undergone. The inferiority of the American product under transverse strain is, according to the Author, as 51 to 87. This fact, he remarks, may account for the opinion of some American authorities that, for constructional purposes, electro-steel is not worth the additional cost. The electric furnace requires a current of roughly 200 to 300 kilowatt-hours per ton of steel, and the total additional cost for current, wages and material may amount to no more than 10s. a ton. Of the several systems of electric furnace now in operation, the Howe appears to offer the greatest advantages over the Martin furnace, on the ground of the absence of included slag, hydrogen and nitrogen in the steel. This superiority rests mainly on the

possibility of obtaining an energetic deoxidation and removal of sulphur. The latter is due to the absence of an oxidizing atmosphere. A number of new forms of electric furnace are undergoing the test of experience.

G. G. A.

Manufacture of Sublimed White-Lead. J. I. BLAIR.

(Engineering and Mining Journal, New York, 1910, vol. xc, pp. 906-8.)

The production of white-lead by sublimation is carried on successfully at two important smelting lead-works in the United States. These plants were designed to convert the flue-dust and the blue fume of the regular lead blast-furnace into the marketable white-fume pigment, basic lead sulphate, thereby ridding the smeltery of a troublesome product. Noteworthy features of this type of plant are its simplicity and efficiency. It has two furnaces of the type known as "slag-eyes," the iron shells of which may be made in any boiler-shop at a comparatively low cost. Their life is about three months. There are six 3-inch tuyeres set about half-way from the top and bottom of the furnace. Directly over these furnaces is a 30-foot brick combustion-chamber, the top of which is covered with zinc cylindrical retorts, 10 inches in diameter and 4 feet long. The circulation of air through these tends to keep them cool. A valuable feature of this method of covering is that the parts may be removed when burned out. Besides a large brick-built fire-brick lined tower 10 feet in diameter and 20 feet high, there are two smaller towers of sheet-iron shells lined with 4-inch red brick, provided with a clean-out door at the base and connected by an inverted "goose-neck." There is, further, a set of seven of these goose-necks, 44 inches in diameter and 25 feet high, made of 14-gauge sheet iron. Each of these rests on a hopper, from which the fume is collected daily. A bag-room containing 540 bags completes the plant. These bags, which are 20 inches in diameter and 30 feet long, are made of unwashed woollen cloth. The natural oil in the wool protects them from the corrosive action of the hot gases.

The fume enters the first tower near the top, and passes out near the base into a short conduit leading to the second tower, after which it issues, by way of the goose-neck, at the top and enters the third tower. From here the fume goes to the goose-necks. As these cool the fume, all the dust and PbO drop out, leaving the pure basic sulphate to enter the bags, into which it is drawn by a 7-foot fan and 50-HP. motor. The pigment collected from the goose-necks, amounting to 3,500 lbs. daily, is of a pink colour due to free PbO . The furnace-charge is composed as follows: Galena, 6 to 7 parts by weight; cuttings, i.e., material taken from the combustion-chamber during the clean-out and consisting of mixture of fused lead sulphate and slag, 1 part; "white-waste" sweepings from the bag-room and the packing department, including such

pigment as may be below standard in colour, 1 to $1\frac{1}{2}$ part; "iron," i.e., boiler-scale, sweepings from machine-shop lathes, and tin cans, the latter serving to keep the charge open, 0.9 part; limestone, purchased in the form of "fines" to pass through a 40-mesh screen, 1.4 part; "black-slag," slag from the slag-eye, rich in lead, 1 to 3 parts; "blue fume," from the blue-bag house, 3 parts; "ashes," or settlings in the trails near the blast-furnace and open hearths, 2 parts; "grey slag," from the open hearths, 2 parts; and coke, 6 to 8 parts. This charge, when mixed, is fed continuously into the furnaces in small shovelfuls, and in a manner to make it fall evenly over the glowing mass, the surface of the charge being kept at a white heat. This high temperature is necessary to ensure not only immediate volatilization of the lead compounds, but complete oxidation of any combustibles drawn over by the powerful draught. A strong inrush of air through the feed-doors furnishes the oxygen needed for the formation of the PbO_2 and PbO . It is important to keep the temperature of the first tower as near to $800^\circ C.$ as possible, for in no other metallurgical operation within the Author's knowledge does the temperature of the furnace so greatly affect the quality of the product.

G. G. A.

Mineral Resources of the Japanese Empire.

Dr. A. SELWYN BROWN.

(Engineering Magazine, London, 1910 and 1911, vol. x1, pp. 346-64 and 568-83.)

After a general summary of Japanese progress since 1874 and the characteristics of the country and its people, followed by a detail of the Mining Bureau, which is modelled on Australian lines, the mining laws, methods and machinery are described, with three Tables of statistics covering 1900 to 1909. The article then specializes, starting with copper and gold, explained by numerous illustrations and Tables.

In the second article, coal, silver and lead, zinc, iron, antimony, manganese and tin, sulphur and petroleum are dealt with, the first-named at the greatest length. The list is not exhaustive, many other products being referred to without details.

C. O. B.

Bronzes and Copper Alloys. A. RZEHULKA.

(Annalen für Gewerbe und Bauwesen, Berlin, 1910, vol. 67, p. 227-34.)

The properties of metals which render them of value in the arts are dealt with in two groups. The first of these includes the so-called working capabilities, such as readiness of melting, toughness,

brittleness, malleability, elasticity, tenacity. In the second group are comprised the commercial qualities, i.e., price, colour, lustre and resistance to mechanical wear, to the chemical action of the weather, gases and liquids, as also the degree of conductivity for heat and for electricity. If in the case of certain metals some of these properties are deficient, it becomes possible by suitable alloys with other metals to alter these characteristics and to adapt them for the purposes desired. It is pointed out that metals vary greatly, in the power of forming alloys, and the facts which govern their formation are discussed, as also the conditions involved in the preparation of eutectic solutions. Copper is the most important metal in view of its power of constituting alloys, and it is distinguished from all the other metals by its peculiar red colour. The properties of copper are dealt with in detail, and it is shown that the specific gravity may be 8.2 to 8.9 according to its relative degree of purity, the melting-point being 1,081° to 1,084° C. Copper bronzes are classed in three groups—those formed by combination with tin, with aluminium and with manganese. The alloy with 17 per cent. of tin is the toughest of the tin series, and in this respect it is about one-third better than pure copper, while in point of hardness the alloy with 20 per cent. of tin gives the best results in this group. A full account is given of the uses of tin bronzes for trade purposes; also of more complex alloys with phosphorus and silicon and those containing a percentage of zinc or lead. The mode of preparing and working the various alloys is described. The aluminium bronzes here noticed are stated to be those in which copper is largely in excess, and it is shown that the addition of 5 per cent. of aluminium to pure copper doubles its tensile strength. All the alloys of copper and aluminium, including "diamond-bronze," with 10 per cent. of aluminium and 2 per cent. of silicon, are discussed. Manganese bronzes, which in practice are those wherein the copper is greatly in excess, are dealt with in a similar way and the properties of many special varieties of bronzes, such as brass, bath-metal, delta-metal, dura-metal, pinchbeck, white brass and bronze-powders, are investigated. Allusion is made to the colouring of bronzes by heating with oil, tallow, paraffin or wax.

G. R. R.

Evolution of the Bulk-Cargo-Steamer. J. E. GIRAUD.

(Le Génie Civil, Paris, 1911, vol. lviil, pp. 199-229.)

The cost of sea-transport may be divided under three heads:—(1) Interest and sinking-fund on original outlay; (2) Working charges while the vessel is actually at sea; and (3) Cost of loading and discharging when in port, which, of course, must include all standing charges while this work is being carried out. Of the first little need be said. The second relates in a great measure to coal-

consumption, which in its turn depends principally on the speed of transit, and has exercised the attention of builders since the advent of the steamer. But as regards the third, the enormous saving to be effected by cheaper and more rapid handling of cargo does not seem to have been recognized in its just proportions until much later; indeed it was not until 1892, when the first "turret" was built, that the importance of a clear hold free from intermediate decks began to be appreciated. Since then the general tendency has been towards the enlargement of the hatch and the elimination of all transverse beams and vertical stanchions. In order to replace the stiffening thus lost, the fore and aft girder or wide waterway-plate placed halfway up the side of the hold was at first introduced, but it was soon abandoned in favour of stiffer frames, as it involved more trimming and interfered with the discharge. A sheet of eighteen cross-sections accompanies the article, and among these are found the various forms of "trunk deck" and "turret," including the double-skin type and the cantilever design, in which the upper wing-tanks are used for ballast. In the text are two cross-sections of "turrets," in which the lower longitudinal corners of the hold are blinded with sloping plates, so as to reduce to a minimum the shovel-work required when the discharging is almost finished. In the second part of the article the American "freighters," which have so completely ousted the old "whale-back," are described, and it is pointed out that, although admirable in their place, this class of vessel is not adapted for deep-sea work. The more modern systems of stiffening are discussed, and the article concludes by bringing forward the advantages of the oil-motor, for which, possibly in conjunction with sails, the Author predicts a great future.

I. C. B.

Sulzer-Diesel Reversible Marine Oil-Engine.

(Schweizerische Bauzeitung, Zürich, 1910, vol. lvi, pp. 228-30.)

This article describes the improvements recently made by Messrs. Sulzer Brothers in the marine type of Diesel oil-engine and contrasts the efficiency of this type with that of the ordinary marine steam-engine.

In 1904, the *Venoge*, cargo-steamer, built by Sulzer Brothers, was provided with a four-cycle Diesel oil-engine instead of the usual steam-engine, reversing being effected by a somewhat cumbrous electrical device. Two years later the firm showed a two-cycle Diesel marine engine at the Milan Exhibition which was afterwards fitted into a steamer plying on Lake Geneva. This type has been considerably improved, and now one of 210 HP. is in use on the icebreaker *Fortschritt* at Hamburg, one of 100 HP. on the Thames and two of 400 HP. of the four-cylinder type on the *Romagna* in the Adriatic trade. The Author gives details of the size and

accommodation of the last vessel, which carries passengers and attains a speed of 12 knots.

An interesting reconstruction has recently been carried out which affords a comparison between a steam-engine and a Diesel engine. The *Schwalbe* (renamed *Uto*) passenger-steamer, 99·4 feet in length, 13 feet beam, with a draught of 3·94 feet and 62 tons displacement, plying on the lake of Zürich, had its steam-plant replaced by a Diesel plant. The old steam-plant developed 70 B.H.P. and the vessel carried 200 passengers with a speed of 8·75 miles per hour. The contract required that with the new plant the vessel should have a speed of $11\frac{1}{4}$ miles per hour. Messrs. Sulzer installed a 150-HP. Diesel marine engine, weighing 9·75 tons, the total weight of the old 70-HP. steam-plant being 14·7 tons, while the length of the engine-room was reduced by 4·26 feet. Tests of the vessel with the new plant twice over a course of 15 miles showed an average speed of $12\frac{1}{2}$ miles per hour, with a cost for oil-fuel of 1·6d. per mile, whereas the cost of coal with the steam-plant was 4·5d. per mile at a speed of 8·75 miles per hour. The weight of coal used was 30 lbs. per mile and the weight of oil 5·9 lbs. per mile. Further improvements in mechanism have been made in the *Romagna* above referred to.

E. R. D.

Tests of a High-Speed Crude-Oil 5-HP. Diesel Engine.

Professor F. ROMBERG.

(Zeitschrift des Vereines deutscher Ingenieure, Berlin, 1910, vol. liv, pp. 1897-1901.)

These tests were instigated by the German Fishery Combine to determine the suitability of such engines for their fishing-smacks. The motor was tested by means of a Prony brake at full load for 8 hours, and further tests were made running light and at one-quarter load, half-load, three-quarter load, and overload, each of these latter tests lasting half an hour. The oil-consumption, speed, indicated and brake horse-powers of the motor, the temperature and quantity of the cooling water used, the temperature and percentage of carbon dioxide, carbon monoxide and oxygen contained in the burnt gases, and also the pressure and the temperature of the air were continuously recorded. The power-consumption of the air-pump could not be measured, but was estimated from the data obtained on larger engines. Galician crude-oil of 0·865 specific gravity and 18,600 B.Th.U. calorific value was used. The following are the chief figures resulting from the tests:—

(1) The mechanical efficiency was 43 to 66 per cent., being 66 per cent. at full load; (2) The oil-consumptions per horse-power-hour were: at full load 0·54 lb., at overload 0·60 lb., at three-quarter load 0·576 lb., at one-half load 0·695 lb., at one-quarter load 1·075 lb.; (3) The overall thermal efficiency was 13 to 25·9 per

cent., being 25.9 per cent. at full load; (4) The consumption of lubricating oil was 0.035 lb. per HP-hour, but could be reduced to 0.013 lbs. per HP-hour, if the engine were run for longer periods.

The chemical analyses disclosed the entire absence of carbon monoxide, a suitable excess of oxygen being present in the burnt gases throughout the tests. The motor ran smoothly and started readily even from the cold.

C. J. G.

Electricity in Waterworks-Pumping. CHARLES B. BURDICK.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 1-19.)

This Paper points out the limitations of electric pumping and indicates the circumstances in which it is profitable. Practice during the last 10 years has tended rapidly towards the use of alternating current, though direct current is still used for distances up to 2,000 or 3,000 feet. Alternating-current motors are usually of 200 to 400 volts, and on long distances the voltage employed approximates to 1,000 volts per mile. Centrifugal pumps are preferred. The following Table shows the relation between the size and the efficiency of such pumps:—

Size of Pump.	Delivery.		Mechanical
	Gallons per Minute.		Efficiency.
Inches.	U.S.	Imperial.	Per cent.
2	98	82	38
3	220	183	45
4	392	327	52
5	881	734	60
6	1,567	1,306	64
18	7,932	6,610	77
32	25,000	20,833	80

The efficiency of the whole system, from engine-shaft to water-delivery, is given as 33 to 63 per cent.

The waterworks of the industrial town of Gary are owned by a company interested in a steel plant about 1 mile distant, at which many thousand horse-power are generated from blast-furnace gases. The plant consists of four single-stage centrifugal pumps, each of 5,000,000 gallons (4,200,000 Imperial gallons) capacity per 24 hours. Two of the pumps are used for the domestic supply. They work against a total lift of 75 lbs., and are driven by 250-HP. motors. The other two pumps, driven by 150-HP. motors, are used for fire purposes, taking water from the domestic pumps, and delivering it at a pressure of about 125 lbs. The new waterworks at Niagara Falls, N.Y., supplying about 14,000,000 gallons per day (12,000,000 Imperial gallons), are operated by electricity transmitted about

4 miles from the large generating-stations. The power is transmitted at 10,000 volts and transformed down to 400. The present price is 36s. per horse-power per year of 8,760 hours, measured at the generating-station, equivalent to about 52s. per horse-power at the pumping-station.

The use of electric power is also justified where water has to be pumped from a number of scattered sources, or where the area supplied contains a number of districts above the general level.

A. J. M.

Pitting of Centrifugal-Pump Impellers. HENRY L. LYON.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 20-21.)

At Buffalo, N.Y., a 25,000,000-gallon (21,000,000-Imperial gallon) vertical centrifugal pump, driven at 500 revolutions per minute by a 1,500-HP. motor, showed a falling off in capacity after about 6 months' service. It could not be spared till the end of a year, when the capacity had fallen off about 20 per cent. It was then taken apart, and the impellers were found to be badly pitted, some of the blades being worn nearly half off. The head during most of the time was 200 feet.

A. J. M.

Physical Phenomena in Centrifugal Pumps. L. BERGERON.

(Revue de Mécanique, Paris, 1910, vol. xxvii, pp. 305-20.)

The mathematical analysis of the action of the centrifugal pump assumes the existence of liquid filaments of infinitely small section. This conception has no relation to the actual case, therefore theoretical results must be accepted with reserve and be controlled by experiment. It is shown that the velocities of the liquid particles situated at any moment at the same radial distance from the centre of the pump cannot be equal, and that they increase from one vane to the other in the direction of rotation of the pump. The consequence is that the usual velocity-diagram of the water issuing from the pump should be drawn with an angle greater than the geometrical vane-angle and with a relative speed greater than that determined theoretically; these increases cannot, however, be determined mathematically at present, and they must be based on the results of experiment. The same considerations are applied to the vortex-chamber, and a curve is given to enable its best shape to be determined.

H. R. S.

Winnipeg Fire-Service Waterworks.

Lieut.-Colonel H. N. RUTTAN, City Engineer.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 150-64.)

The city of Winnipeg has recently installed an independent waterworks system for fire-extinction purposes, with a view to the reduction of the excessive fire-insurance charges which prevailed in 1905.

The pumping-station is situated near the heart of the city, on the bank of the Red River, from which the supply is taken. It was essential that two independent sources of power should be available. A gas-producer plant was therefore installed, with city gas as a stand-by. The equipment of the pumping-station comprises four Crossley two-cylinder tandem gas-engines of 540 HP. each, and two of 250 HP. each, an air-compressing starting plant in duplicate, one producer-gas main supplying all engines, and one city-gas main in reserve. There are four Crossley-type producers:—two of 500 HP., each 8 feet 6 inches in diameter by 18 feet high, and two of 1,000 HP., each 11 feet in diameter by 18 feet high. This plant has an overload capacity of 50 per cent. and uses anthracite, lignite, or any non-caking bituminous coal. The coal is unloaded into a hopper holding 100 tons, and raised by an elevator with rotary feeding-gear to a horizontal conveyer placed above the producer-hoppers. There are four air-superheaters, two 18 feet 10 inches by 5 feet 3 inches, and two 18 feet 10 inches by 4 feet 3 inches, with gas- and air-connections, and dust-collectors. Each unit has a hot gas tubular boiler. The gas from the producers passes in succession through four wet scrubbers, 30 feet high by 5 feet in diameter, filled with coke; three centrifugal tar-extractors, 5 feet in diameter, and two sawdust scrubbers 9 feet by 22 feet. There are also centrifugal circulating pumps and Root air-blowers. A gas-holder of 250,000 cubic feet capacity holds a reserve of gas (producer or city gas) sufficient to operate the whole plant for $1\frac{1}{2}$ to 5 hours. The distributing system comprises 8 miles of lead-jointed cast-iron mains, 10 to 20 inches in diameter, and eighty-five fire-hydrants connected to the mains by 8-inch flanged pipes. The ordinary working-pressure is 300 lbs. per square inch, and the mains were tested, after laying, at 500 to 700 lbs. Each hydrant has four $4\frac{1}{2}$ -inch hose-nozzles with independent gates. The nozzles are reducible to $3\frac{1}{2}$ or $2\frac{1}{2}$ inches, $3\frac{1}{2}$ inches being the largest hose now used. With bituminous coal of 14,000 B.Th.U. per lb. the contractors guaranteed to deliver to the engines 75 per cent. of its heat value. With bituminous slack and coke of 11,442 B.Th.U. per lb. 66·3 per cent. of its total heat was delivered to the engines. The guaranteed capacity of the whole plant is 12,960,000 gallons per 24 hours (10,800,000 Imperial gallons). The actual mechanical efficiency of the pumping unit is 78·40 per cent., and of the engine alone 88·40. The cost of the work was about £188,800.

A. J. M.

Fort William Water-Supply. H. S. HANCOCK.

(Transactions of the Canadian Society of Civil Engineers, Montreal, 1910, vol. xxiv, pp. 173-240.)

Fort William lies on the north bank of the Kaministiquia River, which flows into Lake Superior at the extreme north-east boundary of the town. The river provides a channel 300 feet wide and 22 feet deep. Six miles south of the town and surrounded by high hills is Loch Lomond, 10 square miles in area. This lake is 332 feet above the level of Lake Superior, and its shore line is, for the most part, steep and rugged. The Author describes the construction of the water-supply system derived from this lake, and also that of a temporary supply drawn from a small lake which served until the main works were completed. The works comprised a pipe-line and rock-tunnel to carry the water from the lake to a concrete reservoir of approximately 800,000 gallons capacity. The floor of the reservoir was of concrete not less than 6 inches thick on the rock section and 12 inches on soil and gravel; the walls were 18 inches wide at the top, battering 1 in 12 on the inside face and 1 in 6 at the back. The temporary supply from the small Crescent Lake was admitted through an 8-inch cast-iron pipe. For a distance of 10,600 feet from the reservoir the main was 18-inch wood pipe. This pipe developed serious leaks at the joints, and it was surmized that the dry climate rendered wooden pipes less satisfactory than on the Pacific coast. The iron mains crossed the River Kaministiquia, two trenches, 15 feet wide, having been dredged for the purpose. The Author describes fully the various difficulties encountered in the work, particularly the trouble experienced with the wooden stave-pipe and the methods of overcoming it. The Paper is illustrated by drawings and photographs.

R. S. B.

Esopus Cut-and-Cover Section of the Catskill Aqueduct.¹

(Engineering Record, New York, 1910, vol. 62, pp. 508-511.)

This article describes the self-contained movable plant used on contract No. 11 of 6½ miles out of the 92-mile connection between the Ashokan Reservoir and the distribution-basin for the New York City supply. The equipment comprises mixer, crusher, store-house and conveying machinery for delivering the concrete to the side wall and arch forms, and is incorporated in a single large movable plant which runs along tracks in the trench, the sleepers of which are shaped to fit the finished invert on which they are placed: 7,580 feet of the aqueduct is being built by this appliance, the rate being 45 to 60 feet per day.

C. O. B.

¹ See also Minutes of Proceedings Inst. C.E., vol. clxxxii, p. 416.

Removal of Iron in Organic Combinations from Water.

JOHN R. DOWNS.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 220-26.)

In 1907 two filter-plants of the mechanical-pressure type were installed in the Panama Canal zone, each consisting of three 500,000 U.S.-gallon units, giving a united capacity of 1,500,000 gallons (1,250,000 Imperial gallons). Both supplies were obtained by impounding small streams. Ancon and Panama obtain their water from the Rio Grande Reservoir, and Cristobal and Colon from the Brazos Brook, with filters at Ancon and Mount Hope respectively. Monthly average analyses of both waters are given, the chief differences being in the albuminoid ammonia and iron. The Brazos Brook water yielded 0.355 part of albuminoid ammonia per million, or nearly double that from the Rio Grande water, and contained usually 2 to 3 parts of iron, the largest quantity in the Rio Grande water being 0.8 part.

The Ancon filters deal with the Rio Grande water with ease, running at times considerably above their rated capacity. The Mount Hope filters clogged to the point of completely stopping the flow in a run of 1 to 2 hours, the pressure being 65 to 70 lbs. The water had some odour, but its bacteriological character was good. Aeration removed the odour, but failed absolutely to precipitate any iron. Lime, iron, permanganate and copper were tried as precipitants, but failed completely, sulphate of alumina being finally adopted. The iron, being held in combination with the organic matter, required a large amount of coagulant, no precipitate whatever being formed until 1.2 to 1.5 grain per U.S. gallon (1.44 to 1.8 grain per Imperial gallon) had been used. The volume of the precipitate then produced was incredible. The settling basin was 100 feet in diameter by 7 feet deep, and was divided into eight sections, through which the water, after the addition of the sulphate, passed in succession. At the end of a week the precipitate stood 6 feet deep in the first two and a half sections, having an area of about 2,500 square feet. In the remaining sections there was a uniform layer about 1 foot deep. The precipitate is slimy, and easily flows out with the water through two 10-inch drains. After 4 days' continuous working the organic deposit seems to give off some odour, and a decided nuisance arises if the deposit is left in contact with the water for more than a week. Experiments with a circular tank, having an inverted conical bottom sloping at 30°, go to show that the precipitate can be forced out by the pressure of the water, without emptying the tank. It is important that the full dose of coagulant be added at once, and that the mixture be agitated immediately, an interval of so little as 30 seconds being almost fatal to success. Since the basin has been in operation no trouble has been experienced with the filters. The filtered water is uniformly free from iron.

A. J. M.

2 F

Action of Softened Water on Metals and Alloys.

W. F. MONFORT.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 181-92.)

The water-supply of St. Louis is drawn from the Mississippi River, about $5\frac{1}{2}$ miles below the mouth of the Missouri. After treatment with sulphate of iron and lime it passes through six sedimentation-basins of a combined capacity of about 150,000,000 U.S. gallons (125,000,000 Imperial gallons), approximately two days' average consumption.

The following Table shows the important components, after evaporation, of a representative sample during the low-water period, when the alkalinity was 120 parts per million :—

Calcium carbonate	60.5
Calcium sulphate	13.9
Magnesium carbonate	55.0
Sodium chloride	34.2
Sodium sulphate	85.0
	273.0 (<i>sic</i>)

In solution these compounds are probably dissociated.

When softening was first adopted an attempt was made to carry it to the limit by the use of a large excess of lime, frequently resulting in a very high caustic alkalinity, and producing heavy deposits in the pipes. Lime is now used more moderately, but the formation of the "protective coating" still goes on. The softening process goes on slowly, even at high temperatures, and in winter it is not complete when the water passes into the mains. The softened water acts electrolytically upon metals. In the case of steel and iron the minute traces of impurity in the metal set up differences of potential which accelerate the natural solvent action. Galvanizing affords little real protection, owing to the rapidity with which the zinc is dissolved. Thus the zinc coating of a galvanized cast-iron cup in a new meter, through which 128,266 cubic feet of water passed in 24 days, was almost entirely removed, or converted into zinc oxide and basic carbonates. A destructive action is also exerted upon the gear trains of the meters, its intensity in this case being due to the use of at least three distinct alloys containing widely different proportions of zinc. In nine cases out of ten when the meters failed to register it was found that the gears had completely collapsed. This action is less noticeable in the case of those meters which are set when the river is high and its saline contents low, and therefore receive a thin film of calcium carbonate before they are exposed to the attack of the more corrosive water. Neither iron nor zinc is protected by a coating of tin. Commercial aluminium reacts too readily with even moderately saline waters to warrant its exposure in working parts.

The Author considers it important that only homogeneous materials be placed in contact in such a water.

A. J. M.

Sterilization of Water. WALDEN and POWELL.

Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 128-37.)

The Authors describe an ozone sterilization-plant installed at the Herring Run Station of the Baltimore County Water and Electric Company. The plant, which has an ultimate capacity of 10,000,000 gallons (8,333,000 Imperial gallons) per 24 hours, comprises a raw-water chamber, aspirator-heads, ozone-chambers, ozone-generator and mixing-cell, the travel of the water through the latter being horizontal as well as vertical, to ensure a long contact with the ozone. The generator consists of boiler-plate, shell and head, 30 inches high, 33 inches in diameter, containing each 109 2-inch aluminium tubes, inside of which are micanite tubes or dialectrics (*sic*), containing other tubes resting on a tray, which is connected to one terminal of a special 10,000- to 15,000-volt static transformer, the voltage of the primary circuit being 2,000. The other terminal, which is a ground terminal, is connected to the boiler-plate shell. A continuous violet discharge takes place between the inner and outer tubes, and generates 80 to 100 grammes of ozone per kilowatt (*sic*). The concentration is 5 or less, according to the condition, velocity and temperature of the air. Sparks must be prevented, as they generate nitrous oxide. High temperatures must also be provided against. The generator described will be cooled by oil, the only trouble being the difficulty of making joints which will remain tight even under a pressure of 2 or 3 lbs. of oil. The generators require about 18 watts per square foot of electrode surface at 60 cycles, and 21 watts at 100 cycles. With the atmosphere at or below zero the production of ozone seems to be greater than at higher temperatures. A Table is given showing the number of bacteria in the raw and treated waters respectively, the greatest reduction being from 1,840 to 6, and the least from 1,960 to 1,500 (presumably per cubic centimetre). The lesser reductions are regarded as being due to moisture on the tubes. The colour of the water is appreciably reduced by treatment with ozone, but its turbidity remains unchanged.

At Avalon, Md., sterilization has been effected by means of hypochlorite of lime (bleaching powder) containing on the average 34·2 per cent. of available chlorine. A 0·5-per-cent. solution was used and applied to the water through a graduated orifice, prior to its filtration through rapid mechanical filters. Hypochlorite of lime was found to be more efficient in conjunction with alum than when employed alone; and the use of 0·087 grain per gallon of hypochlorite (say 0·1 grain per Imperial gallon) rendered it possible to reduce the alum from 0·87 to 0·58 grain per gallon (say 1 to 0·7 grain per Imperial gallon). Experience showed also that the use of hypochlorite of lime rendered the water practically sterile, with the total elimination of *B. coli*, and a reduction in colour from 5 to 15 per cent. It increased the lengths of runs between changing the filters, and reduced the quantity of water required for washing them

from 4.1 per cent. to 2.9 per cent. Hypochlorite of lime, even in large quantities, causes but a slight reduction in organic matter; and its bactericidal efficiency is impaired by a high turbidity.

A. J. M.

Sterilization of Water by Ultra-Violet Rays.

V. HENRI, A. HELBRONNER and MAX S. DE RECKLINGHAUSEN.

(Comptes Rendus de l'Académie des Sciences, Paris, 1910, vol. cli, pp. 677-80.)

In a previous communication¹ the Authors described an arrangement whereby large quantities of water (up to 125 cubic metres per hour) could be sterilized by means of ultra-violet light with an energy expenditure of 36 watt-hours per cubic metre. In that arrangement, however, only one-third of the light was utilized, and they now describe a new arrangement which utilizes three-quarters of the light. A lamp of the Westinghouse-Cooper-Hewitt type was placed in a box having quartz windows, and this box was surrounded by a semi-circular cast-iron vessel through which the water flowed, which was fitted with diaphragms, so that the water was not only well agitated but had to pass three times along and in contact with the quartz windows. A by-pass valve was placed on the supply side and held closed by an electric solenoid so long as the current passed through the lamp. If the current failed the valve opened and the flow of water was diverted. The results of working this apparatus for 6 weeks are as follows:—25 cubic metres of water were passed through the apparatus per hour, and the expenditure of energy for light was 26 watt-hours per cubic metre. The water before going through the apparatus contained 30 to 300 germs per cubic centimetre and 50 to 1,000 *coli* per litre. The sterilized water contained 1 germ per centimetre and no *coli*.

H. R. S.

Slow Sand-Purification Works for the New Springfield, Mass., Water-Supply. C. R. Gow.

(Journal of the Association of Engineering Societies, Boston, Mass., 1910, vol. xlv, pp. 189-280.)

Raw water from a river is carried through a 54-inch steel pipe to an impounding settling-reservoir of 10 acres area, from which it is drawn off through a 42-inch pipe to the purification-works. The settling-reservoir, formed by an earthen dam across a narrow ravine, contains 33,000,000 gallons when full. The water then passes through an aerator 50 feet in diameter, and is then distributed to

¹ See Minutes of Proceedings Inst. C.E., vol. clxxiii, p. 420.

the filters. There are six filters of the slow sand type, each 0.5 acre in area. The pure water is collected in 24-inch drains laid under each filter, and thence it flows through the regulator-house to the town. The filter-floors are inverted groined arches 6 inches thick in the middle and 12 inches at the pier-points, which points are 13 feet between centres each way. The concrete was dumped direct into place in the floor. The walls of the filters are about 10 feet high, 20 inches thick on the top, and 29 inches at the bottom, while the division-walls between the filters are 39 inches at the bottom. The piers supporting the roof are 20 inches square, and are arranged on 13-foot centres in either direction. The floor of the filters is covered with gravel 12 inches thick arranged in three layers, over which the filtering sand 3 feet thick was placed. Split tile drains allow the filtered water to drain away. The Author describes the construction of the screening- and washing-plant, by which the sand required for the filters, after being quarried at a neighbouring site, was separated from the gravel it contained and graded into various sizes. The contractor's plant is described in detail, with records of cost for different parts of the work. The Paper is illustrated by drawings and photographs.

R. S. B.

*Operation of Washington Filtration-Plant.*¹ E. D. HARDY.

(Proceedings of the American Society of Civil Engineers, New York, 1910, vol. xxxvi, pp. 1560-1620.)

This filtration-plant has now been in continual operation for 5 years, and the Author describes the results obtained as showing the efficiency and cost of filtration on an extensive scale. The plant consists of a pumping-station for raising the water from a reservoir, twenty-nine filters of the slow sand type having an area of 1 acre each, and the filtered-water reservoir having a capacity of 12,500,000 gallons. There are three centrifugal pumps of 33,000,000 gallons per day each pump against a head of 21 feet, and each sand-washer pump has a capacity of 21,000,000 gallons a day. The filters contain, on an average, 40 inches of filter-sand and 12 inches of gravel. The water-supply for the city is taken from the Potomac River about 16 miles above the city, and on its way to the filters it passes through three settling-reservoirs. The water is rather turbid, due to fine particles of clay. The results are given in extensive Tables showing the cost of operation, extent to which impurities were removed from the water, and comparisons of different systems of filtration. The Paper is illustrated by drawings of parts of the apparatus.

R. S. B.

¹ See Minutes of Proceedings Inst. C.E., vol. clxvii, p. 469.

Cement Water-Mains at Rahway, New Jersey.

WILLIAM BISHOP.

(Proceedings of the American Water Works Association, Charleston, S.C., 1910, pp. 217-19.)

This Paper is of exceptional interest in view of the length of time which the pipes have been in use. In 1871 and 1872 12.6 miles of cement main, 4 inches to 12 inches in diameter, were laid down. Each section is 7 feet long and consists of a sheet-iron shell riveted lengthwise and lined inside and out with $\frac{1}{2}$ inch of cement. Apart from the cement, the iron shell appears not to be coated. The larger pipes are butt-jointed, with a short band at the joint. The smaller sizes are tapered, the small end of one pipe being inserted about 4 inches into the large end of the next. Future house-connections are provided for by inserting a "spud" with lock-nuts in the shell, and filling and casing it with cement. The cost per foot of the mains originally laid, including cutting, filling and making good, was as follows: 12-inch pipes 8s. 7d.; 10-inch 6s. 9d.; 8-inch 5s. 6d.; 6-inch 4s. 5d.; 4-inch 3s. 5d. In paved streets the cost is 10 per cent. more. The cost of repairs, including replacing about 500 feet of main through low ground with iron pipes, averages about £2 12s. per mile per annum. The city is comparatively level, and the ordinary pressure in the mains is 40 to 45 lbs. per square inch. Previous to 1906 the water was pumped direct into the mains; since then it has been pumped into a standpipe. Sudden changes of temperature seem to have more influence on breakages than the difference in the mode of pumping. The pipes are absolutely free from incrustations or deposit, and uniformly clean. Except where the outside coating of cement has been cracked, the sheet-iron shell is as bright as when it was laid.

A. J. M.

Reinforced-Concrete Rising-Main on the Bonna System.

E. R. MATTHEWS, Assoc. M. Inst. C.E.

(Concrete and Constructional Engineering, London, 1910, vol. v, pp. 891-900.)

An account is given of a rising-main, 36 inches in diameter and 4,500 yards in length, constructed in concrete on the Bonna system for the city of Norwich. Tenders were obtained for the work in steel pipes at £12,000 and in cast-iron pipes at between £17,000 and £18,000, while the reinforced-concrete pipes cost between £9,000 and £10,000, inclusive of specials, laying and jointing. The main was not tested before use, but since laid the pipes have been regularly pumped through by reciprocating-pumps and also by Roturbo pumps—the working-head, including friction at the greatest rate of delivery, being 120 feet, according to Mr. Arthur E. Collins,

M. Inst. C.E., the City Engineer of Norwich, who considers them quite safe at 300 feet head. The pipes can be laid and jointed more rapidly than iron pipes with run and caulked lead joints, and, in the case of straightforward runs, thirty pipes were laid per day. A full description is given of the mode of jointing by reference to diagrams. The pipes, each 10 feet 3 inches long by 36 inches in diameter and 3 inches thick, were cast vertically and weighed 30 cwt. each. They were formed on a collapsible drum as a core, and were reinforced with two spirally-wound bands of + section wire in two different sizes, the amount of metal in the outer spiral being sufficient to resist the whole of the bursting stresses. The method of winding the outer reinforcement round a tube of thin sheet-steel, the joints of which were welded by the oxy-acetylene or oxy-hydrogen blow-pipe, is explained, and the inner smaller-section spiral was wound round the collapsible drum. The outer side of the mould or copes were clamped tight round the core. The longitudinal reinforcement also consisted of steel + section wires of the smaller section used for the inner spiral, attached to the latter by wire ties at about each alternate intersection. The joint was made by a reinforced-concrete sleeve which fitted over the ends of the pipes. The method of moulding and laying the pipe is shown by photographs of the actual work. The concrete consisted of one part of cement to three parts of flint and gravel sand from a very fine grade up to about $\frac{5}{16}$ inch in diameter. No crushed material was employed. The main has been in regular use, without the slightest leakage, for 9 months.

G. R. R.

Stresses in Tubes. R. T. STEWART.

(Journal of the American Society of Engineers, New York, 1910, vol. xxxiii, pp. 47-54.)

The Author shows from theoretical considerations that the stresses in the walls of a tube exposed to external fluid-pressure can be represented by a formula similar to that applicable to long columns fixed at both ends. The formulas he obtains are:—

$$\text{For tubes: } S = 42,640 - 127 \cdot 4 \frac{l}{r}.$$

$$\text{For long columns: } p = 86,670 \frac{t}{d} - 1,386$$

$$\text{and } S = 708,000,000 \left(\frac{r}{l} \right)^3 \text{ when } \frac{r}{l} \text{ less than } 0 \cdot 0045,$$

$$\text{and } p = 50,210,000 \left(\frac{t}{d} \right)^3 \text{ when } \frac{t}{d} \text{ less than } 0 \cdot 24,$$

where p is the pressure per square inch to which the tube is exposed, t the thickness of the wall, and d the diameter of the tube,

S the axial load on the column in lbs. per square inch, l its length, and r the least radius of gyration. The plotted results of these formulas give curves, on either side of which the Author's experiments on the collapsing of tubes and many results of experiments on long columns lie quite closely.

H. R. S.

Undermining of the Gigny Dam. HENRI TAVERNIER.

(Annales des Ponts et Chaussées, Paris, 1910, vol. xlvii, pp. 69-76.)

The Gigny Dam, on the Saône between Châlon and Tournus, is of the Chanoine type. In this dam sheet-piling was originally driven both on the upper and the lower sides, but notwithstanding this a serious "blow" under the foundations took place suddenly in November, 1908. The article deals with the method of effecting a repair.

The cavities cut out were very extensive and caused a serious transverse crack to form in the middle pier. An attempt at grouting was first made by introducing cement into the upstream end of the cavities; but although the head of water was purposely reduced as far as possible, the cement was washed right through, and this despite the fact that the downstream openings had been choked with broken stone and gravel held in place by weighted metal plates. This want of success led to the adoption of a new method in which the cavities were fed from the upstream end with gravel, of which no less than 209 cubic yards were rammed into place before any appeared at the downstream end. A wide apron of concrete, 1.6 foot in thickness, was next laid both above and below the dam, the total surface covered in this way being 1,058.5 square yards. These aprons were laid by means of a vertical telescopic shoot 2 feet in diameter supported between two floating-pontoons. On the aprons being completed, and the ends of the cavities thus sealed, holes 3 inches in diameter were drilled at frequent intervals, and down these grout was injected under air-pressure. The pressure at first employed was 43 to 57 lbs. per square inch, but as the work went on it was found expedient to reduce this to about 14 lbs., as otherwise there was danger of bursting the masonry. The holes first drilled were in the pier, where the work was comparatively simple, but subsequently they were put down through the sills, which were of course always submerged. The drilling in the latter case was done from an overhanging gantry attached to a barge. For the grouting 724 tons of cement were used. The cost of this class of work, everything included, is given as £4 8s. per ton of cement.

I. C. B.

Arched Masonry Dam at Las Vegas, New Mexico.

C. W. SHERMAN.

(Engineering News, New York, 1910, vol. lxiv, pp. 446-7.)

After a general description of the requirements, reasons are given for the choice of the design, which differs from ordinary American practice, showing that either a gravity section straight dam or a hollow reinforced-concrete dam of the Ambursen type would be inferior in cost and suitability to the curved plan. The structure has been computed, the particulars being given, as a pure arch, without allowance for such additional stability as the dam may have, acting as a wall. The contract allowed of tenders being sent in for concrete or for rubble masonry construction, the former being ultimately decided upon. The wall, which is 50 feet high, is 4 feet thick at top and 13 feet 11½ inches at base, set to a radius of 250 feet. The foundations are extended down-stream so as to admit of a subsequent raising of the dam to a height of 95 feet. Illustrations and a Table of particulars of twenty-two arched dams in various parts of the world are given. The flash-boards and gate-house are also described.

C. O. B.

Reservoir-Construction in Germany. ROLOFF.

(Zeitschrift für Bauwesen, Berlin, 1910, vol. lx, pp. 555-74.)

The first attempts to create artificial reservoirs in Germany date back to the middle ages, while in connection with the mining operations in the Harz mountains many dams were built across the valleys in the middle of the sixteenth century. In this district as many as seventy so-called ponds (*Teiche*) were formed with a total capacity of about 2,200 million gallons; the largest of which, known as the Oderteich, contains 374 million gallons. Many similar works were carried out for the mines in the Freiberg district, but in other places little was done to store water until the latter half of the last century. No undertaking of any magnitude can be named until the year 1890, which witnessed the construction of the first great embankment in the valley of the Eschbach, near Remscheid. There are now thirty artificial reservoirs in Germany, with a total capacity of 29,150 million gallons which have been constructed at a cost of about £2,850,000, while twenty more are in course of erection or contemplated, for the storage of 105,600 million gallons. These reservoirs serve mainly for water-supply or power purposes in connection with electric distribution systems. Three of the above thirty reservoirs are situated in Saxony, two being near Chemnitz and one in the vicinity of Plauen. Among the projected undertakings several of great importance are in Bavaria,

while others are in the Black Forest in Baden, in Oberhessen and in the Eifel and Harz districts. In the Kingdom of Prussia there are twenty-six reservoirs, eighteen of which are found in the Rhine Provinces and Westphalia, seven in Silesia and one in the Harz. The main facts relating to these twenty-six undertakings are set forth in a Table, and sketch maps are given of the reservoir-sites in the valleys of the Wupper and Ruhr and of those in Silesia. Details are appended of the mode of forming the embankments, with sections of those in the valley of the Urft near Gemünd and in the Neyetal near Wipperfürth. In order to guard against danger to life and property from imperfect design and construction, official regulations for reservoirs drawn up conjointly by the Ministers of the Interior, of Trade and Commerce, of Public Works and of Agriculture, Domains and Forests were promulgated in May 1907.

G. R. R.

Sewer Alterations entailed by Construction of Underground Railway. H. BERGER.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxxiii, pp. 861-3.)

The works of the Berlin Underground Railway have rendered changes necessary in the new Schöneberg drainage works, and caused serious difficulties in construction in consequence of the need of uninterrupted working, the pressure of the subsoil-water, the existence of insecure foundations and the great depths to be attained. The operations involved lateral displacement of stoneware pipes, passage under the railway-lines with flattened sections of sewers and resort to inverted siphons. Siphons had to be introduced at four different places, and a flushing-pipe with a diameter of 11·8 inches and a stoneware sewer of 19·68 inches, were carried in the same size in iron pipes beneath the railway tunnel as siphons, and fitted on the approach side with flushing-slucices which conducted into flushing-chambers, in order that the siphons might be periodically cleansed. The egg-shaped collecting-sewer, 4·97 feet high by 3·28 feet wide, was carried in wrought-iron welded pipes in the form of a siphon beneath the tunnel in the Münchenerstrasse, and flushing-chambers with sluices and ventilators were provided. In the case of the fourth of these siphons, the passage of a large collecting-sewer for the Friedenau district under the tunnel, the difficulties were severe, and the work had to be done in winter when there is least water. The main sewer was diverted into two circular wrought-iron welded pipes, which, in order to discharge the maximum volume of sewage-water, had to be each 4·92 feet in diameter, and were so placed that the left pipe served for conveying the moderately dilute polluted water and rainfall, while the right pipe was used in the case of heavy rainfall only and was laid at a higher level to act as an overflow. Moreover to avoid the acute-angled crossing under the tunnel the direction of the sewer-line was

changed. Previous to the execution of the work the course of the sewer was temporarily diverted to the opposite side to that selected for the siphon. The overflow for the storm-water is 4.5 feet above the bottom of the lower welded pipe. In a chamber on the north-west side is a centrifugal pump, driven by an electro-motor, to pump out the rain-water sewer.

G. R. R.

Volume of Polluted Water in Town-Sewers. G. PRIESTER.

(Gesundheits-Ingenieur, Munich, 1911, vol. xxxiv, pp. 41-3.)

It is usual to estimate the amount of fouled water reaching the sewers by the mean water-supply per head per day, as determined by the municipal waterworks. To this is added the approximate rate of increase and the estimated volume for trade and industrial purposes, and for independent supplies, but this total is in most instances too low, and the Author shows that in the case of Cologne the estimate was more than 4.4 gallons per head too little. It was found necessary, in fact, to double the carefully-assumed mean daily water-supply per head to obtain the total average maximum volume of this part of the sewage-water. In order to demonstrate the facts as to the actual volume of water to be dealt with, the figures in the case of Heidelberg are here examined. For the population on the left bank of the Neckar there are three main sewers which discharge into the river. Careful estimates were made of the bulk of the fouled water, as it was necessary to carry these sewers into one main outfall, and it was originally proposed to conduct the sewage-water by a siphon to the opposite bank of the river, where the new sewage-works were to be situated. The Author was directed to make a detailed estimate of the flow of water in the sewers, the mean amount of fouled water having been assumed at 33 gallons per head and the maximum flow at 49.5 gallons. For the week 18th to 23rd October, 1909, during which the rainfall was slight, the water in each of the three sewers was gauged hourly, and the velocity of the flow was ascertained by flotation experiments. The sewers had been thoroughly cleaned out, and a length of each was selected for the tests where the fall was uniform and there were no inlets. The results, ascertained by means of the floats, were in striking accord with the velocities calculated by the formula of Kutter. The figures are given in Tables; the daily water-supply at this period was 17.6 gallons per head. The population of the area served was 40,000. According to the water-supply at 17.6 gallons, the total bulk of fouled water should have been 684,000 gallons daily, or, on the estimate of 33 gallons per head, as laid down in the conditions, the sewage-flow should have been 1,311,000 gallons, but the actual volume gauged in the 24 hours was 2,622,000 gallons, of which probably 1,938,000 gallons represented trade and industrial consumers. It is apparent, there-

fore, that the sewage-volume is actually double the estimated total in a town of a fairly residential character. Moreover, the results here given for the month of October would doubtless be exceeded during the summer period of maximum water-consumption. The importance of these figures in determining the dimensions and falls of sewers is insisted on.

G. R. R.

Influence of Exuding Gases in Sewage-Sludge. Dr.-Ing. IMHOFF.

(Gesundheits-Ingenieur, Munich, 1910, vol. xxiii, pp. 880-3.)

The manner in which the decomposition and drying of sewage-sludge is affected by the presence of gases contained in the mass is discussed in connection with the deposit produced in the Emscher-brunnen, employed in the tank-treatment of sewage-water. A section of the tank shows how the sludge obtained by simple deposition is allowed to pass into a conical pit, situated beneath the upper funnel-shaped tank, where clarification takes place. It is pointed out that a careful study of the process shows that the behaviour of the gases has an important bearing on the various stages of decomposition. Their action at the beginning is chiefly mechanical in character, as when they first arise they are held for a time entangled in the sludge. When sufficient gas-bubbles have been formed they escape at the surface, but they are often retained so long that the greater part of the mass becomes lighter than water, and this causes portions of the sludge to rise towards the surface. In consequence, however, of the special double-cone construction of the tank the sludge cannot float up into the depositing-tank but tends to accumulate in the surrounding side-spaces, where it forms a superficial layer or coating. In the process of rising to the surface the sludge parts more or less with its gaseous contents and soon sinks by becoming heavier than water; and this operation, constantly repeated, keeps the mass in continual motion, as deeper-lying portions of the sludge fill with gas-bubbles and are carried upwards. In fact, this constant agitation is necessary in order that the biological processes may be duly carried out, and it is important to bear this in mind in designing the plant. In order that the drying of the sludge may take place under good conditions it is necessary that ample drainage should be available under the sludge-tank, as the water escapes beneath. These observations prove the importance of giving free play to the operation of the gases, and of keeping the layers of sludge on the drying-ground fairly shallow, so as to allow the formation of abundant pores in the mass; while in the depositing-tank the sludge must be permitted to accumulate in sufficient depth to give rise to the copious evolution of gases.

G. R. R.

New York Sewerage and Sewage-Disposal.(Report of Sewerage Commission of New York. Submitted 30 April, 1910.)¹

The Metropolitan Sewerage Commission of New York was constituted by the State Legislature in 1906, and reconstituted in 1908, to investigate the present and probable future condition of New York Bay and adjacent waters, and the best means for protecting their purity, and to consider the advisability of forming a sewerage district. The Commissioners were five in number, with Mr. George A. Soper, M. Am. Soc. C.E., Ph.D., as Chairman. Their report is noteworthy alike as regards the magnitude of the problem dealt with, the difficulties encountered, and the painstaking and thorough character of the investigation. The various modes of purification now in use are enumerated, but not dealt with in detail, and their relative merits are not gone into, the Commissioners confining themselves in the main to a consideration of the large administrative, chemical and physical questions involved. The immediate objects of their investigation were: To establish the facts attending the discharge of the sewage, to determine the extent to which these conditions were injurious to the public health, and to ascertain the way in which it would be necessary to improve them in order to meet the reasonable requirements of the present and future. The elucidation of these questions involved a large amount of scientific work, which is described in considerable detail.

Topography and Statistics.—The investigation extended over the territory, some 700 square miles in extent, which drains directly into New York Harbour, and within which the density of population is such as to necessitate a comprehensive treatment of the sewage problem. This territory comprises the whole or part of eighty-four cities and towns in the States of New York and New Jersey, and is referred to as the "Metropolitan District." Its population was 5,332,186 in 1905, and is expected to reach 11,866,400 in 1940. The present daily volume of sewage is estimated at 617,500,000 Imperial gallons, and the anticipated volume in 1940 is 1,787,000,000 gallons, or 150 gallons per head per day. The assumed composition of New York sewage, based on a water consumption of 100 Imperial gallons per head per day, is as follows:—

Residue on evaporation	670	parts	per	million.
Organic and volatile matter	340	"	"	"
Nitrogenous matter	120	"	"	"
Fats, etc.	42	"	"	"
Total carbon	170	"	"	"
„ nitrogen	12	"	"	"

The district is intersected by several waterways, notably the Hudson River and the East River, which divide it into six distinct areas, the conditions in each of which are separately and minutely

¹ A copy of this Report is in the Library of The Inst. C.E.

set forth. By far the most important of these areas is Manhattan Island, which, prior to the annexations of 1873, contained the entire city of New York.

Manhattan Island.—This island extends some $13\frac{1}{2}$ miles from north to south, with an average width of $1\frac{1}{2}$ mile, and is traversed by a longitudinal ridge, highest towards the north. The side-slopes are fairly steep, affording good gradients for the sewers, but along the water-fronts there has been formed a strip of made ground, averaging 8 to 10 feet above mean tide-level. The population in 1905 was 2,102,928, giving an average of 90,000 per square mile; but in the tenement districts of the east side, believed to be the most densely peopled areas in the world, there are more than 200,000 people to the square mile. The large business houses are crowded into the south, or seaward end of the island, and more than half of the length of its water-front is occupied by quays and piers.

Manhattan has been sewered from time to time on the combined system, at an estimated cost of £5,200,000. It has 522 miles of sewers, with no less than 172 distinct outfalls, discharging all around the island, and ranging in size from a 12-inch pipe to the equivalent of a 13-foot culvert. The sewers are mostly of brick, and egg-shaped, a usual size being 4 feet by 3 feet. Many, even of the old sewers, are in good condition, but in other cases the joints have been disintegrated by very hot liquors and acids, and by the exhaust steam which is frequently discharged into the sewers. Ventilation is provided by perforated manhole covers, and disconnecting traps on the house-connections are almost universal. The sewers are not flushed, and the practice is to let the deposits accumulate to a depth of 10 to 20 inches, and then remove them by hand. The amount of these deposits is greatly increased owing to the practice which prevails among the street sweepers [employed by the Bureau of Highways] of pushing and washing street dirt into the catch-basins on the sewers. The Commissioners draw attention to the way in which the City suffers from this and other cases of lack of co-operation between different departments.

Most of the outfalls are tide-locked; and a tide 2 feet higher than normal backs up the sewers over one-fifth of the area of the island. Work on the sewers is rendered difficult by the large number of other pipes and conduits which occupy the streets. A new complication has lately been introduced by the Subways now in course of construction for rapid transit purposes, some sixteen of which will probably be completed in the near future. These are being built near the surface, mostly under the avenues which run north and south, or normal to the main sewers. The Public Service Commission claims the right to reconstruct all sewers met with on the lines of the Subways, and frequently does so without reference to the Bureau of Sewers. In some instances large sewers have been robbed of their fall for considerable distances, and converted into sewers of deposit. To such an extent are the sewers interfered with by the Subways that the Sewerage Commissioners think it may be desirable to reconstruct the whole of the former on the

separate system, providing deep pipe sewers for house-drainage, and large shallow sewers (with inverted siphons under the Subways) for storm-water.

The conditions met with in Manhattan Island hold good to a greater or less extent in other parts of the district.

Disposal of Sewage.—All the sewage of Manhattan, and most of that from elsewhere, is discharged in a crude state into tidal water. In the few cases in which purification is attempted, it is confined for the most part to precipitation with lime, either alone or in conjunction with perchloride of iron, and the deodorization of the sludge with chlorine. The results are generally unsatisfactory. Only in one or two cases is the tank effluent filtered.

Scientific Investigations.—The Commissioners have carried on an extensive series of observations, and applied a variety of tests, designed in the first place to establish the fact and the extent of the pollution. Visual evidence was obtained in the shape of fields of grease, floating sewage matters, and wood, which in calm weather covered several acres, and were sometimes a mile or more in length. Mud examinations, to the number of 1,100, showed that a large part of the bottom of the harbour was polluted with deposits of sewage solids, the origin of which was established under the microscope. The tidal creeks which receive a large part of the sewage of Brooklyn were especially offensive, and odours were met with in various parts of the harbour.

The chemical test relied on was the determination of the dissolved oxygen, the Albert Levy method, employed by Professors Letts and Adeney, being used for the purpose. The process recommended by the British Royal Commission on Sewage Disposal being found to require about 3 hours for the solution of the precipitated iron oxides, modifications were introduced by Professor Metzger which reduced this period to 5 minutes. The amount of dissolved oxygen in the water of the Upper Bay at ebb tide averaged 64 per cent. of saturation, and that of the Harlem River 56 per cent., the Gowanus Canal and Newtown Creek being absolutely devoid of oxygen.

The Upper Bay and the Harlem River each contained an average of 16,000 bacteria per cubic centimetre at ebb tide, and the Passaic River at Newark 141,000.

The mean range of tide in New York Harbour is 4·4 feet, and the average daily discharge of land-water into the Harbour 2,284,588,800 cubic feet. The average of the ebb- and flood-flows through the Narrows is 11,665,000,000 cubic feet per 6 lunar hours. Three series of float experiments were made with three types of floats, and charts are given showing the set of the surface current in each of the various channels at each lunar hour. The average velocity of the swiftest tidal current in the Harlem River is 2·4 feet per second, and in the East River 5 feet.

Dispersion and Diffusion.—A multiplicity of outfalls has been favoured as conducing to a more complete dispersion and diffusion of the sewage; but in practice it is entrapped in the creeks and the spaces between the long piers which project from the shores,

surrounding the city with a field of sewage. A typical calculation shows that the proportion of sewage to clean water in the Harlem River tends to reach 11 per cent., and that in the East River 20 per cent. Experiments were made to determine the rate of diffusion of sewage in tidal water containing varying proportions of land water; with 75 per cent. of land water, sewage rises from a submerged outlet at a rate of 0.3 foot per second, and with 16 per cent. at 0.5 foot. The sewage forms a layer over the tidal water, and may preserve its identity for a considerable time. Observations were also made of the rate of digestion of organic matter in the harbour water, much of the former being completely decomposed in 3 or 4 weeks. Experiments in the deposition of sludge from admixture with water showed that sea water deposited its burden more than 12 times as fast as land water.

Conclusions.—The Commissioners regard it as proved that, contrary to popular belief, the tidal currents do not flush out the harbour satisfactorily, but cause the sewage to oscillate back and forth near its points of origin. Their studies show that the digestive capacity of the harbour is so reduced by pollution as to call for immediate restrictions on the discharge of sewage into it. In particular the proposed construction of trunk sewers to carry the sewage of populous valleys down to the Hudson River and the harbour is viewed with grave apprehension. Not only does the discharge of sewage produce objectionable conditions near the outfalls, but the water in the main channels of the harbour and in the East and Hudson Rivers is more polluted than considerations of public health should allow.

It has not been proved that the discharge of sewage into the harbour has had any appreciable effect on the general health of New York or its suburbs; but it is impossible to avoid the conclusion that bathing in the harbour, collecting driftwood from its shores for fuel, and eating shellfish grown in it, was attended by risk of sickness. Many floating baths are located within a short distance of sewer-outfalls. In six cases the water in these baths was strongly coloured within a few minutes by a dye placed in the sewers. The deaths from typhoid in 1907 per 100,000 were: New York 17.5, London 4, Paris 10, Berlin 4, Vienna 3, Dresden 2.

Recommendations.—The Commissioners were appointed, not to formulate a scheme, but to outline a policy. They were confronted by the initial difficulty that the two halves of their district lay in two independent States, the dwellers in which nevertheless form practically a single community. New Jersey was twice invited to join in the investigation, but without result.

If it should be found impracticable to form an interstate sewerage district, administered by an interstate commission, the reporters recommend that a metropolitan sewerage district and commission for New York State be created without delay.

No system of sewers to bring the sewage of the whole metropolis to a single point being practicable, the metropolitan territory should be divided into sections. An outline plan should be prepared to

which all future sewerage work should conform. A large portion of the sewage now discharged into the Harlem River and into the upper part of the East River should be intercepted and taken elsewhere for disposal.

To a considerable extent purification works, embodying the principles of sedimentation, screening and filtration, should be employed ; but the necessity for discharging crude sewage in certain cases is expressly recognized.

The report covers 550 pages, and is profusely illustrated with maps, photographs, and tidal and other diagrams. Seventeen engineers and technical assistants, in addition to the five Commissioners, were engaged in the investigation, which cost about £18,000.

A. J. M.

Klönne System of Carbonizing-Chambers. PARSY.

(Compte Rendu de la Société Technique de l'Industrie du Gaz en France, Paris, 1910, pp. 175-208.)

The plants at Königsberg, Rotterdam and Padua are described and illustrated. The installation at Königsberg is of inclined chambers, and although these continue to give satisfactory working results, it is now considered advisable to erect horizontal chambers in view of the high initial cost of erection and the difficulty in preventing the clinkering of the material within the inclined chambers unless special coal be used. The working results at Rotterdam and Padua are given in Tables and curves. In carbonizing over extended periods, as in the case with carbonizing-chambers, the question of the variation in the quantity and quality of the gas produced is of special interest, and curves are plotted to demonstrate the fluctuation in yield, calorific power, specific gravity and illuminating power of the gas over the whole carbonizing period. An installation at Rixdorf (Berlin) and one at Frankental (near Mainz) are briefly described, and it is stated that the towns of Padua and Frankental are lighted exclusively by gas from carbonizing-chambers. From the results quoted the Author feels justified in claiming the following advantages for the system of carbonization :—Abolition of night work, allowing a day of 8 hours to be adopted without inconvenience ; Abolition of Sunday work ; Reduction of labour ; Production of gas of better quality to that of ordinary retorts of the same yield ; Abolition of scurfing, of stopped ascension-pipes and of naphthalene ; Liquid tar and of superior quality ; Yield of ammonia augmented about 30 per cent. ; Harder coke and in large pieces ; Reduced cost of maintaining settings.

E. V. E.

Condensation in Coal-Gas Manufacture. GREBEL.

(Compte Rendu de la Société Technique de l'Industrie du Gaz en France, Paris, 1910, pp. 239-255.)

This Paper is based on the Author's knowledge of the Klönne and Otto types of condensers. After deprecating those methods involving the rapid cooling of gas and the contact of cooled gas and cold tar which have been practised for a considerable time, it is pointed out that the most favourable results are obtained by condensing at a moderately high temperature and allowing intimate contact between the gas and the vapours condensed from it, before reducing the temperature to that of the surrounding air. By this means coal-tar vapour, which is acknowledged to be highly efficacious for the removal of naphthalene, is fully utilized for this purpose, by maintaining the tar at a suitable temperature, whilst benzol is not removed to the same degree as by treatment in the cold. A description is given of a Klönne condenser in which tar is caused to gravitate through a series of highly-perforated diaphragms, and, in a finely-divided condition, is brought into intimate contact with the ascending gas. The condensing-towers are of large diameter and the process of cooling the gas, which is effected by the surrounding atmosphere, is gradual and slow. The tar at the base of the tower is maintained at 60° C. (140° F.) to prevent the extraction of benzol from the ascending gas. Within the tower a form of fractional distillation takes place as the result of carefully adjusting temperatures and contact of the tar vapours, which, under these conditions, are capable of completely removing naphthalene. It is usual to find 4 to 7 per cent. of naphthalene in gasworks' and coke-oven tar; a chemical examination of the tar used in Klönne condensers, however, showed the presence of 12.6 and 14 per cent. of dry naphthalene. The Otto system of condensing finds special application with coke-oven installations where the "direct" method of sulphate-of-ammonia manufacture is operated. The principle of gradual cooling and treatment with finely-divided tar is adhered to, the tar, however, in this case being injected against the gas-stream. The Author contends that a less elaborate apparatus might be equally advantageous, and believes that an empty ammonia-scrubber might be utilized. The finely-divided tar might be caused to gravitate against the gas-stream, and the rate and degree of cooling might be controlled by the temperature of the washing-tar.

E. V. E.

Vertical Retorts and Chambers in Germany.

(Bradford Corporation Gas Department. Report of Sub-Committee, 1910.¹)

In view of necessary extensions and alterations of some of the Bradford Gasworks, a Sub-Committee of the Bradford Corporation

¹ A copy of this Report is in the Library of The Inst. C.E.

Gas Committee have reported on a visit made to several of the most important works in Germany, where vertical retorts and chamber-settings are in operation. Several installations of Dessau vertical retorts were visited, the working conditions and results being described. The members were much impressed by the small number of men employed in the retort-house where the actual production of gas per man was 270,000 to 310,000 cubic feet per shift. At Bradford 25,000 cubic feet of gas are produced per man by hand-stoking and 33,000 cubic feet by machine-stoking. The total cost of retort-house labour with vertical retorts was stated to be less than 5*d.* per ton of coal carbonized, whereas the cost at Bradford is 2*s.* 8·98*d.* Two installations of Munich carbonizing-chambers are described. It is pointed out that several other systems of vertical retorts are being developed in England and Scotland, notably the Woodall-Duckham and Glover-West processes of continuous carbonization, an important advantage of which is that one quality of gas can be uniformly maintained, while with chambers, working 24-hour charges, the varying qualities of gas generated have to be mixed.

For purposes of comparison the working results obtained with the various installations, as well as the latest Bradford results, are given. It is necessary to note, however, that different kinds of coal are used with different installations.

Type of Setting.	Gas per Ton of Coal Carbonized.	Candle- Power.	Coal per Charge.	Time in Charging.	Gas made per Man per Shift.	Wages per Ton of Coal Carbonized.
	Cubic Feet.	Candles.	Cwts.	Seconds.	Thousands	<i>d.</i>
Bradford hand-stoked.	10,440	17	2½	120	25	35·51
Bradford machines .	10,622	17	3	45	33	27·81
Munich chambers . .	12,300	12	65	40	..	4·0
Berlin verticals . .	11,300	15¾	11½	30	285	5·0
„ new	13,430	10	21½	..	388	2·5
Sunderland verticals .	12,028	17½	9	..	..	..
St. Helens (Glover- West).	12,400	16	60 per day	..	290	2·75
Bournemouth (Woodall- Duckham)	14,000	14	50 per day	..	..	..

The general conclusions arrived at were, that in the event of a vertical retort-plant being erected at one of the Bradford works in lieu of the existing horizontal retorts, the improved results would effect a saving to the Corporation after paying all charges of interest and sinking-fund.

E. V. E.

Steel Gasholder-Tanks with Curved Sides. E. BONNET.

(Compte Rendu de la Société Technique de l'Industrie du Gaz en France, Paris, 1910, pp. 283-296.)

It is usual to design metal gasholder-tanks, both large and small, in the form of an open cylinder fixed to a bottom plate by angle-iron. With small tanks the drawbacks are not serious. With large tanks, however, the thickness of metal of the vertical cylinder necessary to resist the water-pressure becomes unduly large. The thickness of plates is proportional to the product of diameter and height of the tank, and with tanks of 200 to 250 feet diameter, a metal 2 to 2½ inches thick may be necessary to allow a safe working-limit. Under these conditions the weight of the tank is too great, the work of riveting too difficult, and the price is high. The principle of the tanks manufactured by the Maschinenfabrik Augsburg-Nürnberg is to replace the present vertical rectilinear side, with its unequally distributed strains, by a flexible curved side in which the strains are equal at all points. The form of curve is described by the Author and the strains are analysed. By such a design a considerable reduction in the weight and thickness of metal is effected. As the tanks are designed to carry the full weight of water, special lattice-girder work is allowed to prevent deformation whilst filling. The main weight of the tank is taken by vertical supports, and in constructing foundations the strength of these supports is the most essential. The foundation required for the central portion of the tank is comparatively light. A gasholder of 5½ million cubic feet capacity with a tank of curved sides is in use at the Simmering Gasworks, Vienna, and four gasholders of capacities between 2 and 9 million cubic feet are in course of construction.

E. V. E.

Construction of Aerial Circuits: French Ministerial Decree of March, 1910. MAXIME and ROBERT WEIL.

(Lumière Électrique, Paris, 1910, vol. 12, pp. 335-67.)

The Authors deal with the French Ministerial Decree of 21st March, 1910, as to the construction of aerial circuits, limiting themselves in this article to the provisions which relate to the height and mechanical resistance of conductors above ground. In determining the maximum sag, the factors of temperature and accidental loads or overloads due to wind (horizontal), and ice and snow (vertical), are considered. The Authors conclude that the maximum sag occurs always at the highest temperature and without the above overloads. In France this is about 60° C. The fundamental equations affecting the sag are dealt with mathematically, leading to a determination of the maximum tension. Variations due to overload are then considered, an example being illustrated by curves.

J. A. S.

Open Atmosphere and Dry Transformer-Oil as High-Voltage Insulators. H. J. RYAN.

(Proceedings of the American Institute of Electrical Engineers, New York, 1911, vol. xxx, pp. 1-76.)

The object of this Paper is to ascertain the losses governing the formation of corona in high-voltage transmission-lines, and thus determine the limit of insulation of the atmosphere; the same considerations are applicable to oil-insulation. The information obtained by other experimenters, both in the laboratory and with actual transmission-lines, in particular those of the Central Colorado Power Company, are compared with the experiments made by the Author with concentric cylinders, and in the main a satisfactory agreement is shown. The conclusion comes to is that the corona is due to the ionization by the collision which is caused when electric stress is greater than 76 kilowatts per inch. A certain "headway" is needed for the electrodes to acquire the necessary striking velocity, so that a certain striking distance from the negative conductors is needed to produce the effect. This striking distance varies with the diameter of the conductor and increases almost proportionately to it up to a diameter of 0.075 inch, when it is 0.05 inch. Then it increases more slowly up to a diameter of 0.1 inch, when it is 0.06 inch; after this it increases very slowly, so that at a diameter of 0.5 inch it is 0.07 inch, and the maximum striking distance is 0.25 inch for very large diameters. Partial corona are formed at much lower voltages, depending on various circumstances, such as the antecedent amount of ionization of the atmosphere, the captured ions and radioactive emanations at the conductor-surfaces, and the mechanical form of the conductor-surface. The Author finally obtains the following formula for calculating the line-voltage at which corona-loss becomes serious:—

$$E_{crit} = 455 k \frac{17.9 b}{459 + t} \cdot \frac{d + 2a}{C};$$

where E_{crit} = critical kilovolts (effective sine-wave); $k = k_{pc} \times k_e$, factors selected by judgment guided by the results of practical tests and experience; d = outer diameter of wire or cable in inches; a = corona striking distance obtained from curve in Fig. 2; b = barometer, inches of mercury; t = temperature, °F.; C = capacity of 1,000 feet of line in microfarads.

H. R. S.

Experiments with Hanging- and Pin-Insulators. G. BENISCHEE.

(Elektrotechnische Zeitschrift, Berlin, 1910, vol. xxxi, pp. 1131-4.)

The recent practice of using several hanging-insulators in series for high voltages is discussed and criticized, and amongst others the following disadvantages are cited: greater height needed for the

poles and swinging of the line-wire when blown about by the wind. Several methods of placing pin-insulators in series are described, and it is shown that the arrangement which gives the least capacity offers the greatest resistance both to the invisible brush-discharges and to arcking. Photo-reproductions of such insulators, placed three in series and exposed to pressures of 130,000 to 205,000 volts are given, showing visible brush-discharges at the lower voltage and arcking at the higher. The electrical discharges are clearly shown on these photographs.

H. R. S.

Large-Output Mercury-Rectifiers. B. B. SCHÄFER.

(Elektrotechnische Zeitschrift, Berlin, 1911, vol. xxxii, pp. 2-5.)

Up to the present mercury-rectifiers have not been able to deal with more than 30 to 40 amperes. A modified type is described capable of carrying 250 amperes or more. It consists of a steel cylinder closed at each end by means of grooved flanges. A sheet-iron cylindrical casing is placed outside the steel cylinder and contains oil for cooling. The conductors are taken through the flanges by means of insulated caps, and the inside of the steel cylinder is exhausted. Oscillograph-records of the rectified current are given, and tests show that the efficiency is 92 per cent. with 150 kilowatts, dropping to 90 per cent. at 50 kilowatts, and at 10 kilowatts it is still 87 per cent. The voltage-drop is small.

H. R. S.

Wireless-Telegraphy, Quenched Spark by Geissler Tubes.

MAX WEIN.

(Jahrbuch der drahtlosen Telegraphie und Telephonie, Leipzig, 1910, vol. iv, pp. 135-57.)

An account is given of a series of experiments made by the Author to determine the quenching effect of Geissler tubes inserted in series with a spark-gap. The apparatus consisted of a spark-gap circuit inductively connected to an oscillating circuit, which in its turn was inductively connected to a measuring circuit; the details of the capacities and inductances used are given. Spark-balls made of silver, aluminium, copper and magnesium were used. The spark could be varied between 0.5 and 2.0 centimetres, and the degree of coupling between 14 and 1.7 per cent. The apparatus was tried with and without Geissler tubes.

The principal results are as follows:—Without a Geissler tube the quenching of long sparks is only possible with a loose coupling; silver electrodes have the greatest quenching effect and magnesium electrodes the least. The resonance-curve with the Geissler tube is

displaced towards the longer wave-lengths and shows much greater damping than without the Geissler tube; this is specially the case with silver electrodes. The quenching effect of a Geissler tube does not depend on the shape of the tube, on the nature of the gas, or on the form of the electrodes, but on the pressure of the gas; the best pressure lying between 0.1 and 1.0 millimetre of mercury. The quenching effect diminishes with higher voltages; with 80,000 volts two Geissler tubes in series are necessary, and with 100,000 volts no regularity in the quenching could be obtained under any conditions.

H. R. S.

Electrical Properties of Compound Wires. F. E. FOWLE.

(*Electrical World*, New York, 1910, vol. 56, pp. 1471-5, 1521-5, and 1911, vol. 57, pp. 108-11.)

The Author points out that the necessary requirements of conductivity and tensile strength are not fulfilled for all conditions of service by a homogeneous wire of any known material, and describes the Monnot wire, this being a compound wire of steel core with copper shell. The article is the outcome of a research into the properties of such compound wires, and a comparison of theoretical values with test-results, as regards resistance and inductance. Wires of steel core with copper shell, and steel core with aluminium shell, are taken, and their weights, sizes and conductivities with reference to copper are first given. The mile-ohms of compound wires and of solid copper wires are compared. The inductance and distribution of current between core and shell are then discussed, and the theoretical results are compared with a series of determinations made at the Electrical Testing Laboratories, New York.

J. A. S.

Chemical Specification for Rubber-covered Wires and Cables.

C. R. BOGGS.

(*Electrical World*, New York, 1910, vol. 56, pp. 1234-5.)

The Author points out the want of standard methods of analysis in rubber compounds used on wires and cables, and refers to the specification adopted by the German Association of United Cable Manufacturers and by the German Royal Establishment for Testing Materials, including the chemical details of procedure and interpretation of results. The Author checked the methods in the laboratory of the Simplex Electrical Company, and found the results better than by the methods commonly in use. Details of the tests are given.

J. A. S.

Safety-Device for Overhead Electric Wires.

(Revue Universelle des Mines, Liège, 1910, vol. xxxi, pp. 298-303.)

The necessity for protecting the public against the dangers of a broken live-wire of an overhead tram-line, or high-tension power-transmission line, has brought a number of safety-devices to the notice of engineers. That which appears to be most in favour in continental countries is known as the "Apicea." Any contrivance of this nature must be simple, effective, and cheap, must cause no loss of current, impede in no way the normal working of the line, and be free from the liability of coming into action under any condition short of rupture of the wire. But when a breakage occurs, however unfavourable the conditions may be, such as an accumulation of snow or a violent gale, it must act quickly and with certainty, so as to shunt the current out of the broken wire before the loose end reaches the ground or falls upon a cross-wire. It is claimed for the Apicea device that it meets satisfactorily all these requirements. At the moment of rupture it mechanically short-circuits the current. It consists essentially of a simple lever pivoting round an axis carried by the insulator, which lever is not dependent on a spring or any other piece of mechanism capable of failing to act when required. The source of energy utilized is not the weight of the falling end of the wire—a slow and feeble force—but the horizontal reaction produced by the difference in mechanical tension between the unbroken and the broken wires carried by the post or other support. This force is sufficient to overcome any resistance caused by the lever sticking through oxidation or other cause. The short-circuit is made from the broken wire to one of the other wires in the system, in the case of an alternating current, and with the return rail in the case of tramways, with a single wire, or with two wires of the same potential. The short-circuit with the line wires has the great advantage of preventing, on a power-transmission line, an insufficient earth-connection, while in the case of a tram-line, the connection of the short-circuiting wire with the rail ensures a good "earth." The short-circuit operates a disjuncter at the power-station whereby the current is cut off from all the wires of that section, thereby rendering it impossible to restore the current till the line has been repaired. It has been found in practice sufficient to provide a protector for every third support. It can be seen from a distance when it has come into action.

G. G. A.

Tests of Losses on High-Tension Lines. G. FACCIOLO.

(Proceedings of the American Institute of Electrical Engineers, New York, 1911, vol. xxx, pp. 99-117.)

Corona-losses are of serious importance in transmission-lines using very high voltages, and this Paper gives results of tests made on the lines of the Central Colorado Power Company. The loss at various voltages varying between 80 and 105 kilovolts between the generating-station at Shoshone and Denver (153·5 miles) and between Shoshone and Leadville (63·5 miles) were determined, and show a very rapid increase at the higher voltages. The following Table gives some of the results:—

Kilovolts at Shoshone.	Kilowatts, Shoshone to Denver.	Kilowatts, Shoshone to Leadville.	Difference.
80	109	10·6	98·4
90	341	37·9	303·1
100	1,003	152·0	851·0

These are no-load losses and are mainly due to the exaltation of the voltage towards the end of the transmission-line, which was found to increase 10 per cent. between Shoshone and Denver, so that the corona-loss was concentrated at the Denver end. With a load on the transmission-line the voltage would drop along the line, and the corona-losses would be greatly diminished and would become practically negligible when the voltage at Shoshone was 100 kilovolts.

H. R. S.

High-Tension Cables. P. HUMANN.

(Elektrotechnische Zeitschrift, Berlin, 1910, vol. xxxi, pp. 1265-7.)

The design of cables suitable for working with pressures of 30,000 to 40,000 volts and over is discussed, and the following formula is obtained giving the relation between the radius of the conductor and the thickness of the dielectric:—

$$\log (r + \delta) = \frac{\epsilon}{2 \cdot 3 r} + \log r,$$

where ϵ is the ratio of the pressure to be carried by the conductor to the maximum electric stress in the dielectric, which occurs close to the conductor, and δ is the thickness of the dielectric.

This formula is plotted, and it is thus shown that there is a minimum diameter of the cable, namely, when R is equal to ϵ , a result which can also be obtained by differentiating the above

equation. A similar formula is obtained for three-core cables, and it is then found that the minimum over-all diameter of cable occurs when the radius of the conductor is equal to two-thirds of ϵ .

H. R. S.

Gaucin-Seville Electric-Power Transmission.

(Sociedad española Oerlikon—"Transporte de Fuerza Gaucin-Sevilla." ¹)

The power for this installation is derived from the river Guadiaro at Gaucin, 78 miles from Seville. A minimum of 53 cubic feet per second is taken with 420 feet fall; during 8 months of the year 140 cubic feet per second are available. The dam is 82 feet long, 12 feet high and 2 feet 8 inches wide on top, with slope up-stream of 1 to 5 and of 2 to 5 down-stream. The canal is 6,234 yards long, of which 1,200 yards are in tunnel; the mean width is 7 feet 5 inches, depth 4 feet 9 inches, and gradient 5 per 10,000 in open cut and 1 per 1,000 in tunnel. The water is admitted to the turbines through a reservoir containing 12,336 cubic feet, and two 40-inch pipes 547 yards long, the first 220 yards in reinforced concrete, and the rest, from the point where the pressure exceeds 160 feet, in Martin-Siemens steel, riveted, with steel flanges and india-rubber joints. Three Pelton wheels of 1,500 HP. and 410 revolutions per minute, and two 100-HP. turbines for excitors and lighting, drive three three-phase generators of 1,300 kilovolt-amperes. The tension is 5,000 volts and 41 periods per second, raised by transformers to 52,000 volts. The small turbines drive two continuous-current generators of 65 kilowatts 110 volts at 1,000 revolutions. There are two three-phase transmission-lines, 0.216 inch in diameter, carried on 1,550 steel posts about 90 yards apart, and there are three long spans of 219, 328 and 492 yards where the wire is suspended from steel cables. Distance between wires 40 inches; maximum height above sea-level 2,526 feet at Monte Berrueco, and minimum 36 feet near Seville. Three-phase current is distributed by sub-stations at Villamartin, 34 miles, Utrera, 58 miles, Dos Hermanas, with a branch to Alcalá de Guadaira, 70 miles, and Seville, 78 miles. In Seville the primary tension is 27,000 volts and the secondary 3,500 volts.

C. A. F.

High-Falls Development of the Peshtigo River in Northern Wisconsin.

(Electrical World, New York, 1910, vol. 56, pp. 1227-32.)

This article describes a 66,000-volt equipment transmitting energy over 62 miles to several towns from a hydro-electric station

¹ This pamphlet is in the Library of The Inst. C.E.

of 5,000 kilowatts capacity. The dam and hydraulic works are described, these creating a series of three lakes of 1,670 acres, the total area equalling a storage of 1,254,630 kilowatt-hours with 80-per-cent. efficiency of the turbines. The two-story power-house contains five 1,900-HP. horizontal-shaft turbines driving three-phase generators and the step-up transformers on the ground floor, the switches, etc., being on the second floor. A feature of the arrangement is the use of Swedish-iron conductors for the high-tension bus-bars to increase their inductance to prevent the entrance of lighting. The description includes the high-tension switching arrangements and the transmission-line.

J. A. S.

Flexible-Tower Construction of Taylor-Falls Transmission-Line.

(Electrical World, New York, 1910, vol. 56, pp. 1418-20.)

This is a description, with photographic illustrations, of the first line constructed with new flexible-tower **A**-frames. The spans are nearly 400 feet, and the towers are braced erect by being clamped to a $\frac{3}{8}$ -inch steel earth-wire running from square anchor-towers spaced 1 mile apart. The "prop" towers are built of channel-bars spaced 9 feet apart at the bases, converging to 4 feet 3 inches at a height of 42 feet above ground. A peak on this carries the earth-wire, and a cross-arm carries pin-insulators at each end for 50,000 volts, the third insulator being mounted on the **A**-frame crosspiece. The line-wires are 7 feet apart and 5 feet minimum distance from the earth-wire. The four-legged anchor-towers are also described; 40 miles of transmission-line required 413 **A**-frames 42 feet high, and twelve 36-foot **A**-frames, besides sixty-eight square anchor-towers.

J. A. S.

New Era in Electrical Illumination. R. W. HUTCHINSON, JUN.

(Engineering Magazine, London, 1910-1911, vol. xl, pp. 365-387 and 507-519.)

The first article is devoted to metal-filament lamps and the second to the principal forms of flaming-arc and vapour lamps. The Author points out that the industry has been revolutionized within the last 3 years by the production of such lamps on a commercial scale. After a discussion as to the characteristics of these, including remarks on the question of appliances for reflection and radiation, the Author proceeds to describe and review the several systems of manufacture. With regard to the tungsten filaments, the support, useful life, radiating power, temperature, effect of voltage-variation on life, cost of operation in comparison with carbon lamps, and commercial effect of their introduction are treated. The first article

concludes with the consideration of the factors retarding the rapid introduction of the lamp, its principal field of service and its proper installation. The flaming- or luminous-arc lamp is described in detail in the second article, which concludes with an account of the magnetic luminous-arc lamp due to the investigations of Dr. Steinmetz, of the General Electric Company.

C. O. B.

Improved Mercury-Vapour Lamp. MAURICE LEBLANC.

(Le Génie Civil, Paris, 1911, vol. lviii, pp. 244-6.)

A great improvement has been effected in the mercury-vapour lamp by constructing the envelope in quartz, a substance which, by its extremely high melting-point and its practical immunity from cracking due to heat by reason of its extremely low coefficient of expansion, has made it possible to reduce the luminous length of the lamp from 43 inches to 1·6 inch and the diameter from 0·98 to 0·59 inch. This reduction in size is followed by the production of a much higher temperature and an increased efficiency, while the glowing of the quartz envelope serves to modify the objectionable greenish tinge so noticeable in the light emanating from the original Cooper-Hewitt lamp. The consumption of current per unit of light is only very slightly above that of the flaming arcs, and the fact of there being no carbons to attend to more than compensates both for this and for the high initial cost. The terminals are made of "invar," a name given to that alloy of iron and nickel in which the coefficient of expansion practically vanishes, and the joint between these and the quartz envelope is made by grinding and not by fusion. The lamps last 1,000 to 2,000 hours, and, unless actually broken, can usually be made as new by cleaning and refilling. The movement which it is necessary to impart to the lamp when lighting up is furnished by an electromagnet, and a resistance is placed in the circuit to control accurately the difference in pressure between the terminals. The quartz envelope allows the short or ultra-violet rays to pass through, and where the lamp is used for lighting, as opposed to sterilization or the production of chemical phenomena, these physiologically injurious rays must be intercepted by surrounding the lamp with an outer glass globe. The light is, of course, much less diffused than that obtained from the original Cooper-Hewitt lamp, but is much more so than that given out by the electric arc.

I. C. B.

Luminous Neon-Tubes. G. CLAUDE.

(Comptes Rendus de l'Académie des Sciences, Paris, 1910, vol. 151, pp. 1122-4.)

The Author states that, owing to the unusually low value of the dielectric cohesion of neon, this gas is specially suitable for luminous vacuum-tubes, and whereas a pressure of 3,000 volts is necessary with nitrogen-tubes, equal luminosity can be obtained with neon with 1,000 volts. Minute amounts of other gases have, however, a very detrimental effect, but since neon is less easily liquifiable than the other gases, these latter can be absorbed by means of charcoal at the temperature of liquid air, and a method of manufacturing these tubes, based on the above property of charcoal, is described. The Author has experimented with a neon-tube 6 metres long and 45 millimetres in diameter, and has obtained one candle-power for an expenditure of 0.64 watt, not taking into account the losses in the transformer; adding these losses, the result is 0.8 watt per candle-power. He also found that there was a loss of 350 volts at the terminals, so that with longer tubes a better efficiency would be obtained. The light has an orange tint.

H. R. S.

I N D E X

TO THE

MINUTES OF PROCEEDINGS,

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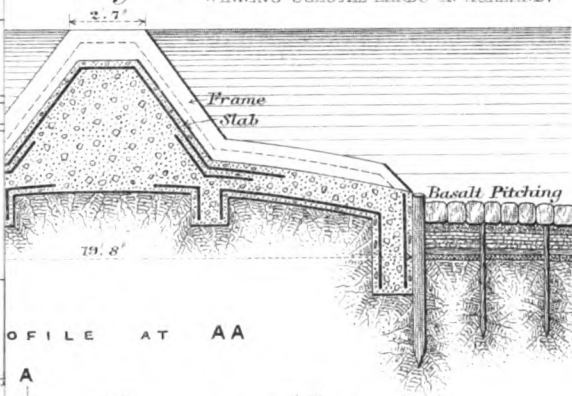
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Fig: 6.

WINNING COASTAL LANDS IN HOLLAND.

OF FILE AT AA



A

8' 0"

4' 0"

SLAB

FRAME

A

ELEVATION AT BB
BREAKWATER

Fig: 9.

DE M

LLE



CONCRETE FACING

50

40

20 F

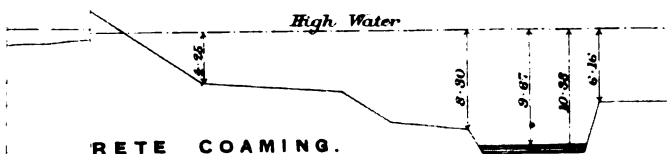
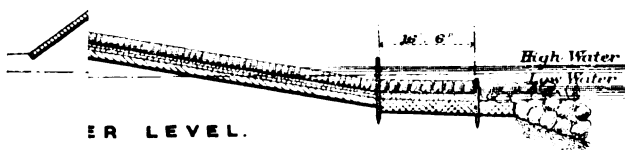
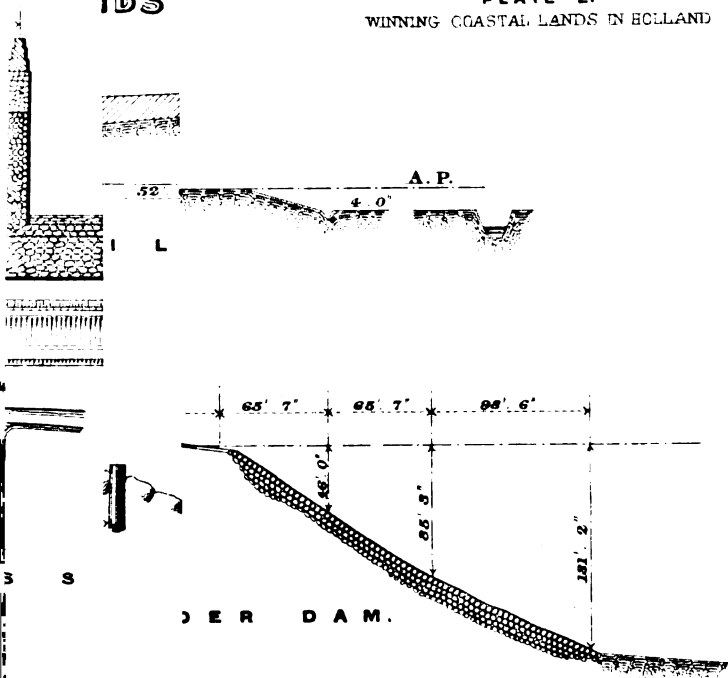
Basalt Concrete Facing
26.3'

ROAD

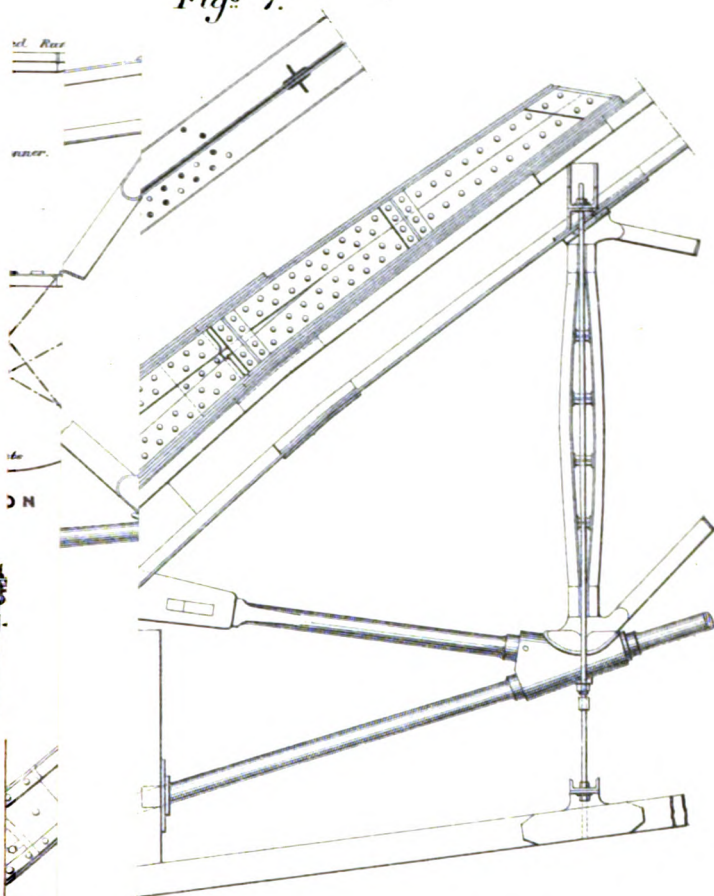
Basalt Pitching

7.30'

WEN



Fig^s 7.



5 TO PRINCIPALS N^os 31 & 32
ON NORTH WALL.

— S C A L E S —

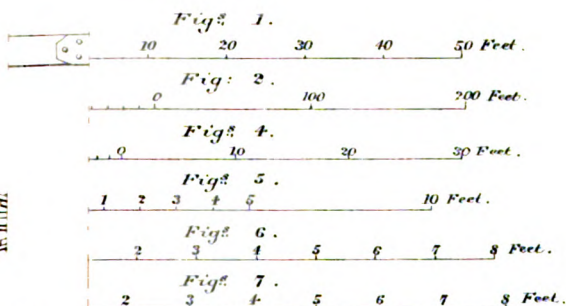
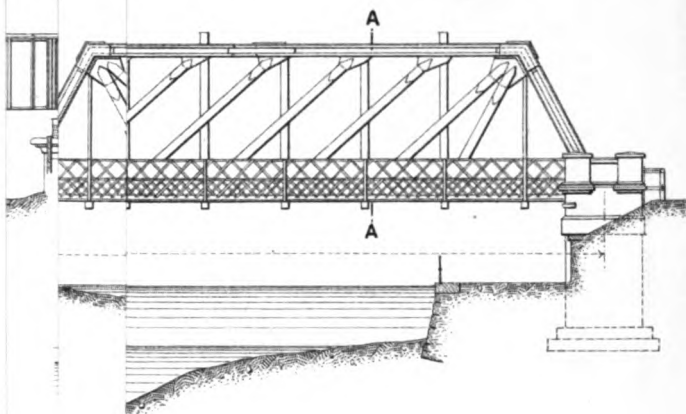
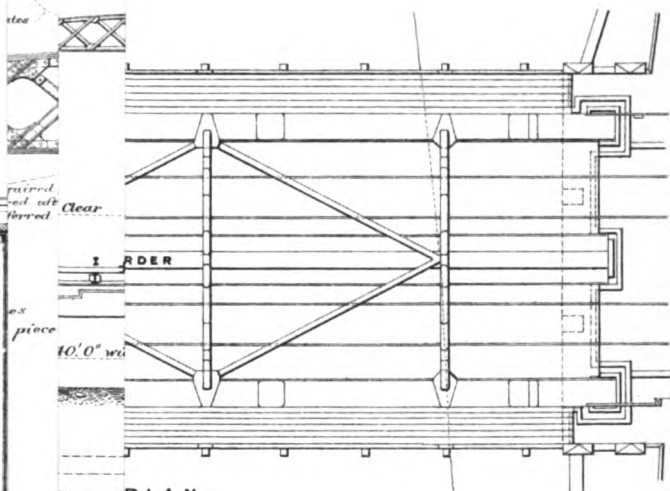


PLATE 4.
ARPLEY BRIDGE, WARRINGTON.

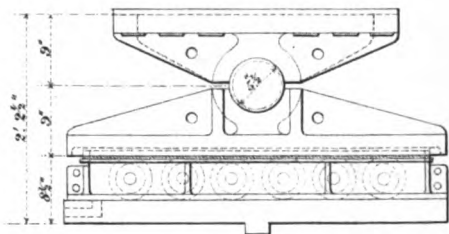


F



PLAN.

Fig 7.



DETAIL BEARINGS.
1/8" in an Oil-Bath.

— SCALES —

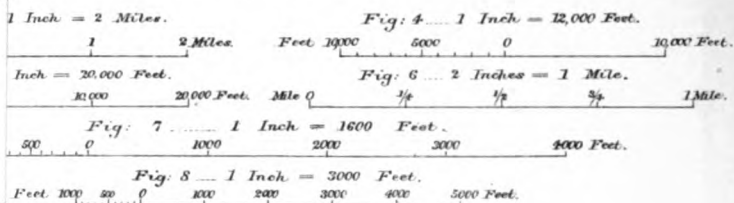
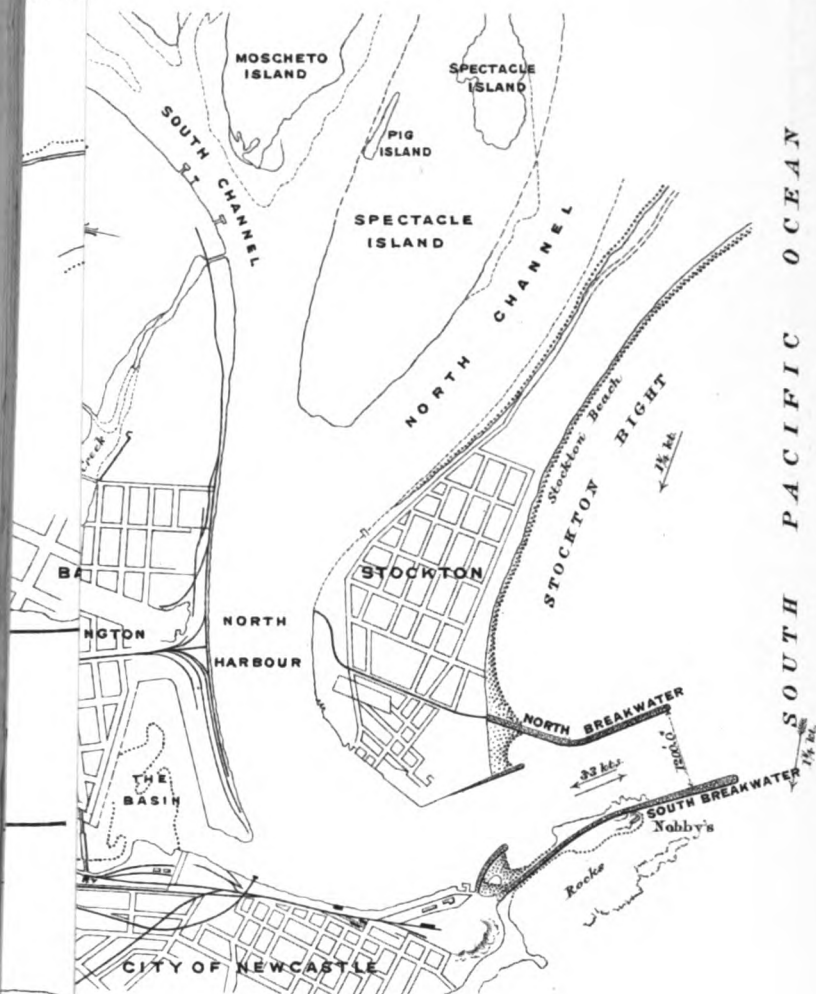


Fig: 8.

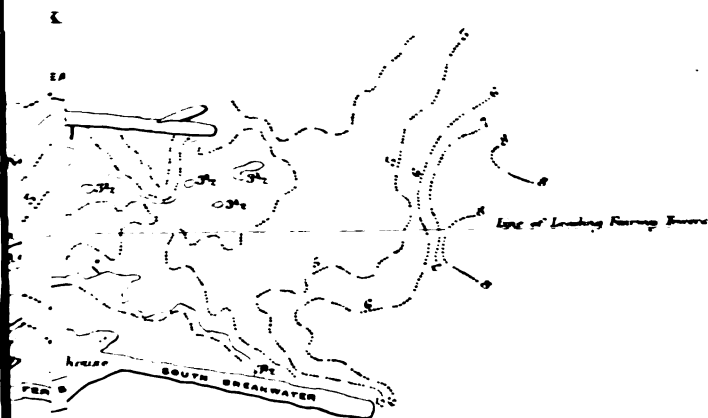


PORT OF NEWCASTLE.

PLATE 6

RANGE V. D. M. N. S. A.
NEWCASTLE ENTRANCE
N. S. W.

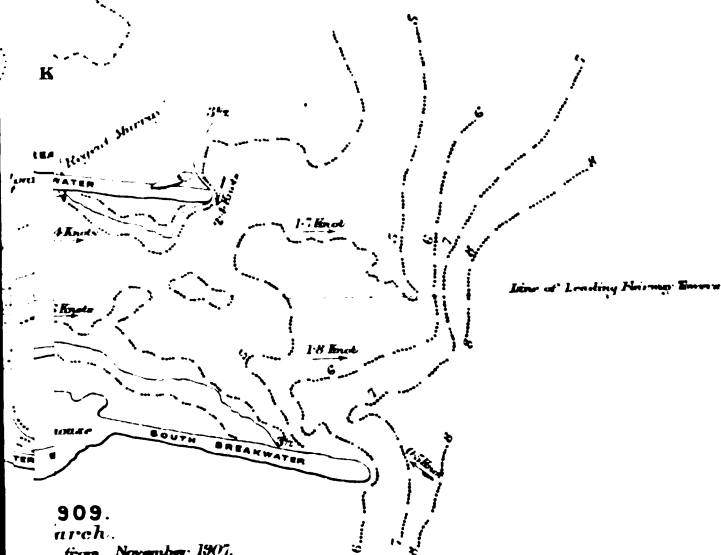
9.



37.
ber.
Nov. 1905 to Nov. 1907.
10 Tons

Fig. 10.

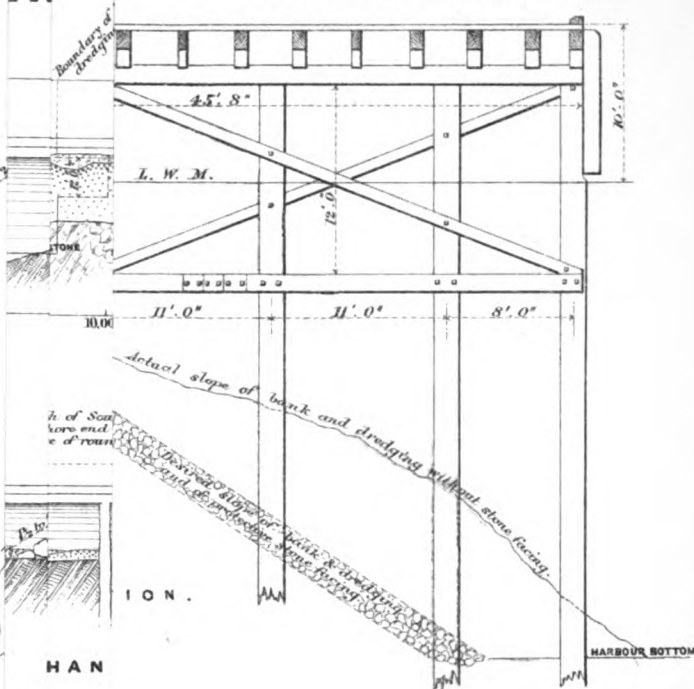
DICKTON BIGHT



909.
arch.
from November 1907.
1050 Tons.

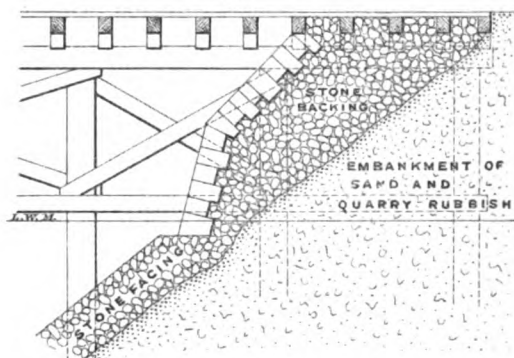
PLATE 7.
FREMANTLE HARBOUR-WORKS.

ER



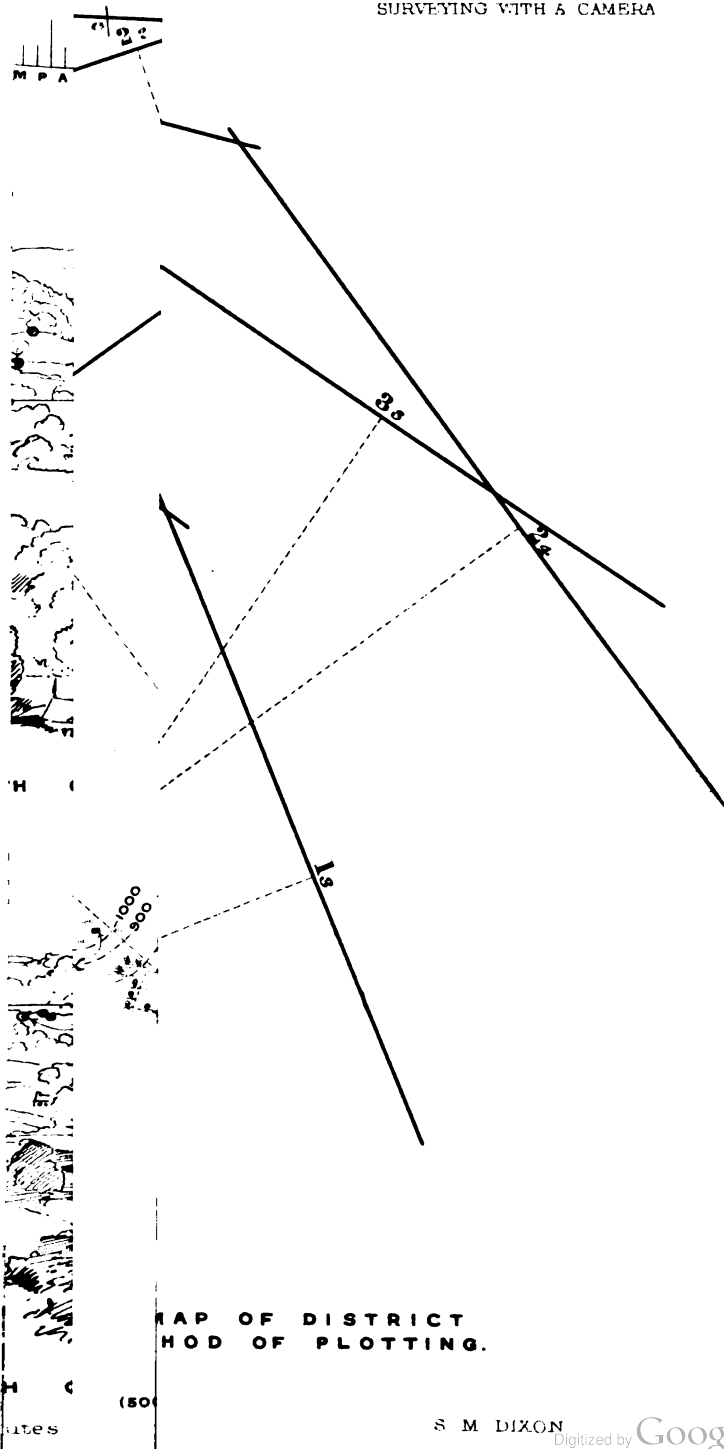
PLAN SHOWING CROSSHEADS.
SOUTH QUAY.

Fig. 8.



STONE WALL TO PREVENT ACCUMULATION OF RUBBISH.
NORTH QUAY.

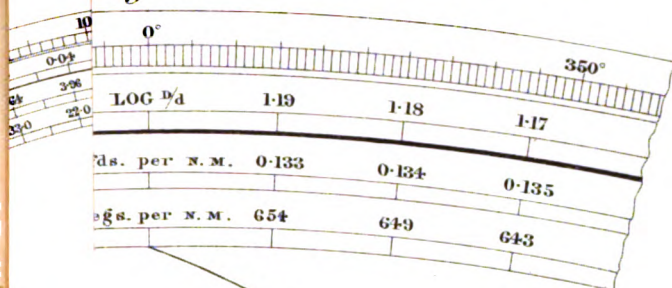
PLATE 8.
SURVEYING WITH A CAMERA



INSTRUCTIONS.

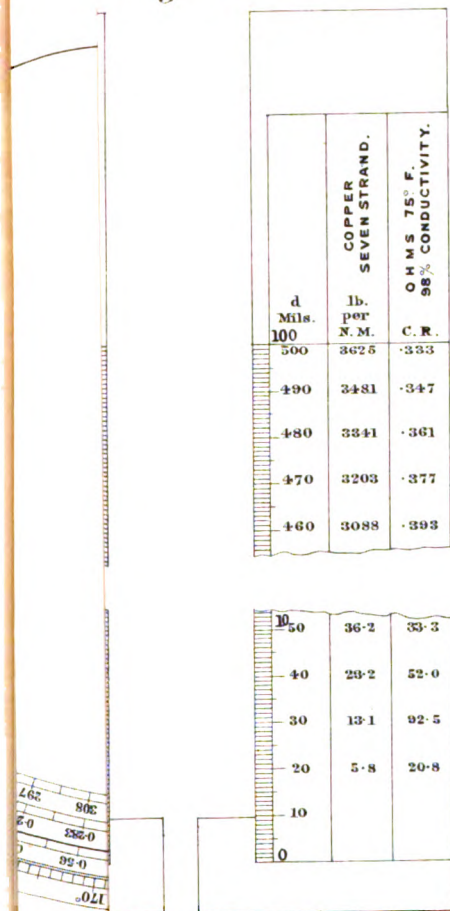
PLATE 9.
INSTRUMENT FOR SUBMARINE-
CABLE AND OTHER CALCULATIONS.

Fig: 2.



ON OF CIRCLE — FULL SIZE.

Fig: 3.

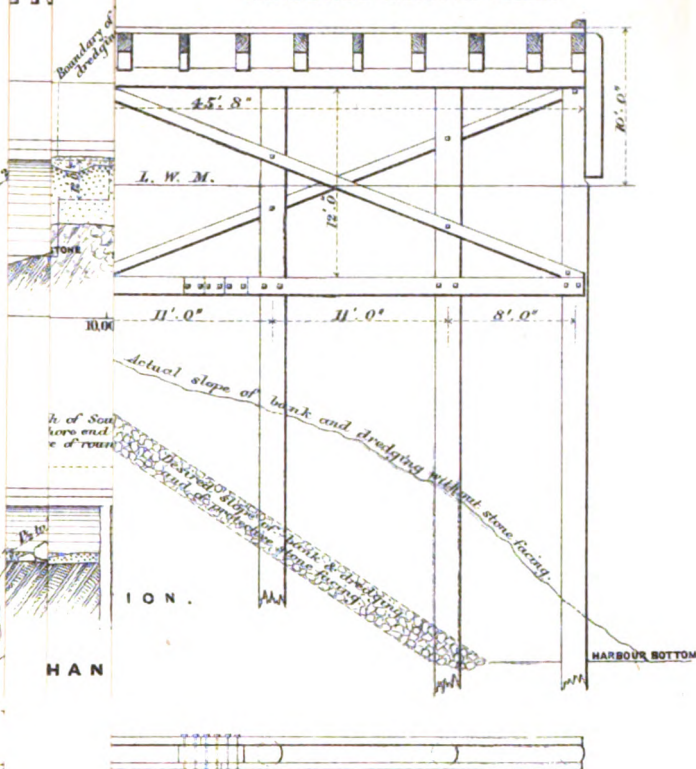


C FOR CONDUCTOR.
DIAL ARMS — FULL SIZE.

B.8.2.22.

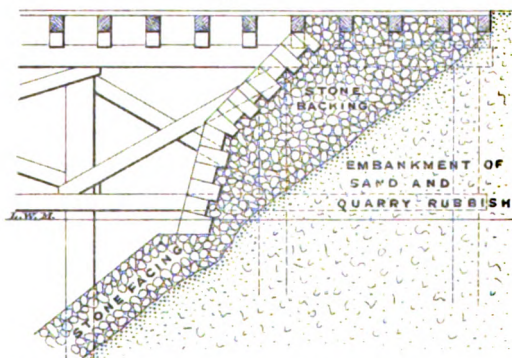
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PLATE 7.
FREMANTLE HARBOUR-WORKS.



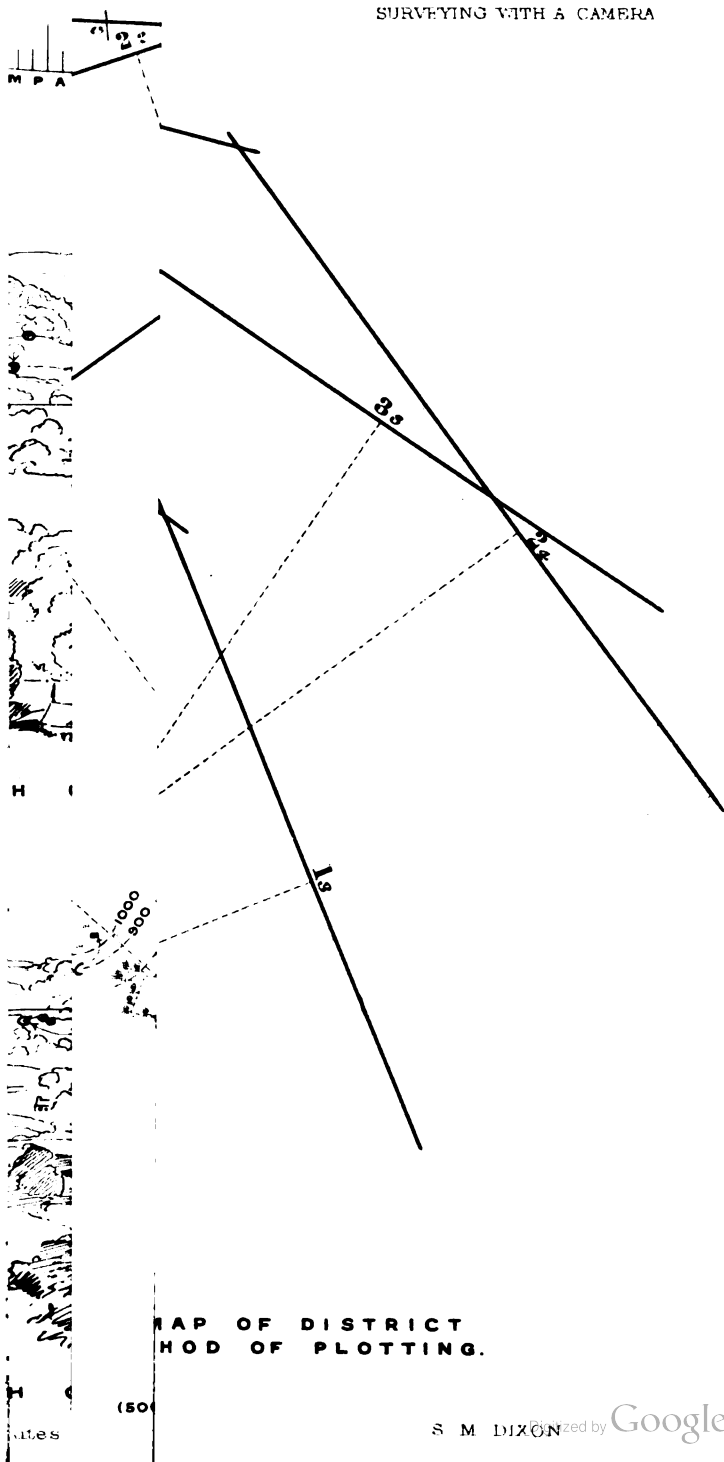
PLAN SHOWING CROSSHEADS,
SOUTH QUAY.

Fig. 8.



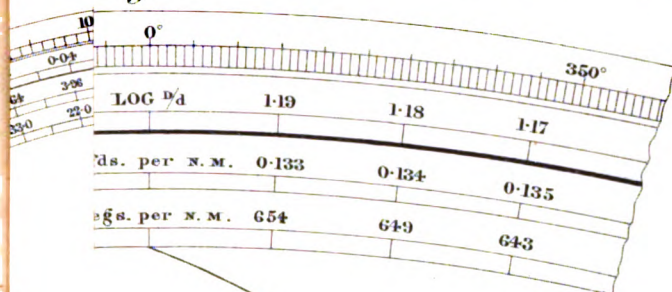
STONE WALL TO PREVENT ACCUMULATION OF RUBBISH.
NORTH QUAY.

PLATE 8.
SURVEYING WITH A CAMERA



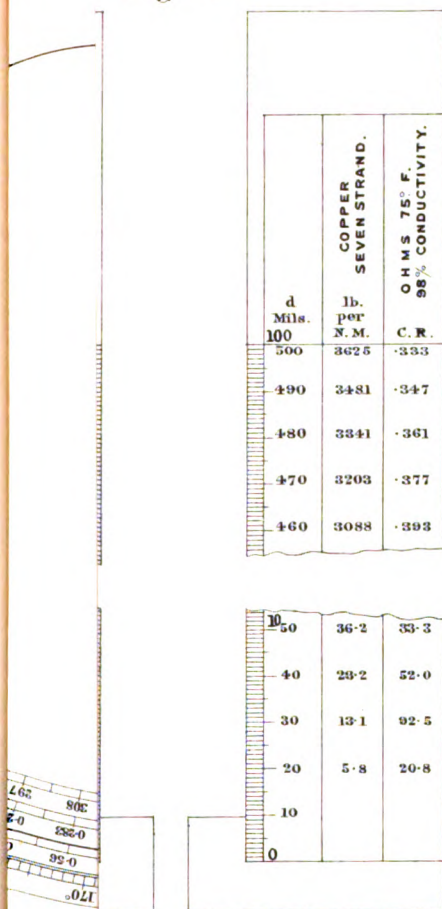
TRUATIONS.

Fig: 2.



ON OF CIRCLE — FULL SIZE.

Fig: 3.



C FOR CONDUCTOR.
DIAL ARMS — FULL SIZE.

PLATE 9.
INSTRUMENT FOR SUBMARINE-
CABLE AND OTHER CALCULATIONS.



Annex is sized 2

Howell
ANNEX
Fall, 1984



Enos B. Sizer

FIVE
ANNUAL
Fall, 1904



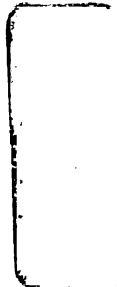
ANNEX B SIZE E

FIVE VOLS
ANNEX
Fall, 1984



Annex to slide 2

THE
ANNEX
Fall, 1964



1

